NEW APPLICATION OF VIBRATION DAMPER TO REDUCE ALONG WIND LOADS

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1. ABSTRACT

This paper presents an unusual practical example of using a damper.

A damper system is generally required for to avoid the consequences of wind or pedestrian induced vibration such as :

a) *To reduce or cancel excessive vibration generated by the vortex shedding (turbulences – Von Karman vortex…) .*

b) *To increase the lifespan of the structure with respect to fatigue.*

c) *To increase the level of comfort for inhabitants of high-rise buildings (increasing the damping would result in reduction of the level of acceleration; acceleration resulting of vortex shedding as above).*

d) *To increase the level of comfort on bridges or footbridges, in buildings when the natural frequencies are in the range to be excited by pedestrians (increasing the damping would result in reduction of the level of acceleration; acceleration resulting from the action of people walking).*

In this paper we present how a damper can help to reduce the along wind action so that to save money both on the structure and foundation costs.

2. CLASSICAL USES OF VIBRATION DAMPER

2.1 WIND INDUCED VIBRATION

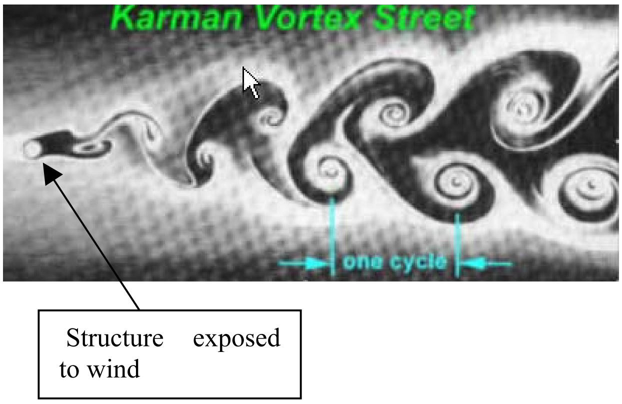


Fig 1 : Karman Vortex Street

When the structure is exposed to wind vortexes are created downstream. The vortexes are created at regular intervals; if the frequency of the vortexes is in phase with one of the structures natural frequency, the structure would start to vibrate.

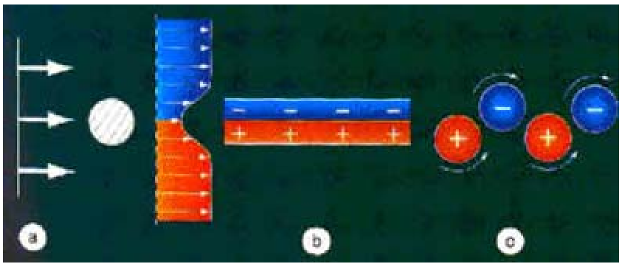


Fig 2 : formation of the pressure area

Alternatively, due to vortexes leaving the structure given areas in pressure or under pressure are generated and consequently alternative forces are acting on the structure.

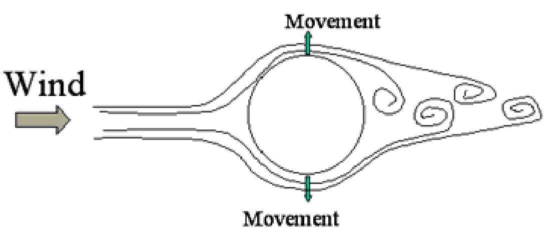


Fig 3 : cross wind vibration

The structure is subject to alternative forces in the crosswind direction and would vibrate in that direction.

The wind speed for which the frequency of the vortex is in phase with one of the structures natural frequencies is called: critical wind speed. If one of the critical wind speed is within the wind speed range in the considered area then structure vibration can be expected.

If the critical wind speed is low such as 10 to 13 m/s, which is rather a low wind speed occurring every day (depending on the area) a high number of vibration cycles can be expected. The structural resistance with respect to fatigue has to be investigated. If the critical wind speed is high then the risk is no longer fatigue but huge vibration amplitudes with extremely important loads (wind loads are a function of wind velocity at the power 2).

Not only slender structures such as stacks, columns, masts and towers are affected by this phenomena. Other structures such as bridges or footbridges are often affected.

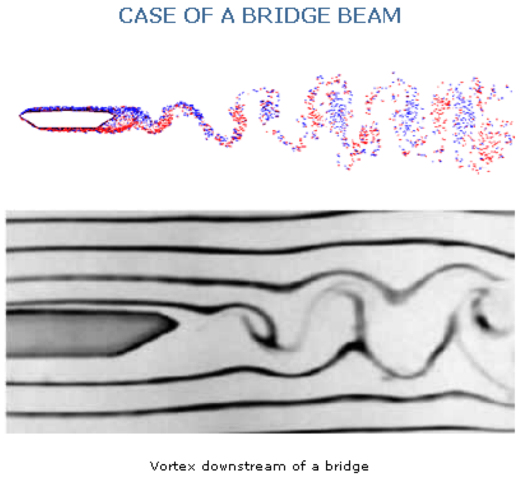


Fig 4 : vortex behind a bridge deck

These two examples show the vortex behind a bridge deck. These vortexes are going to create vertical and/or torsional vibration in the deck. The vibration of the deck could have different consequences such as fatigue problems in the structure, unacceptable amplitude of vibrations with a risk of failure (see Tacoma bridge), sensation of un-comfort for pedestrians with risk of panic.

Fatigue of a structure is dependent mainly on the number of vibration cycles and on the amplitude of vibration. For wind loading the number of vibration cycles can be huge when the critical wind speed is low. There is no way of reducing in most of the cases the number of vibration cycles because it depends on external causes such as wind, walking pedestrians, etc. So the only way to reduce the fatigue stresses is by reducing the response of the structure.

If you consider a structure such as a cylindrical stack, the max amplitude Yc at the top is a reverse function of the Scruton number (Sc).

$Sc = 2 * m * \delta / (Q * d^2)$

m : reduced or modal mass /meter
δ : Logarithmic decrement of damping
Q : density of air
D : outer structure diameter

So by increasing the logarithmic damping δ, we proportionally reduce the structures max amplitude Yc.

A structure can be modelled by individual masses mi vibrating with the frequency fi and the amplitude yi.

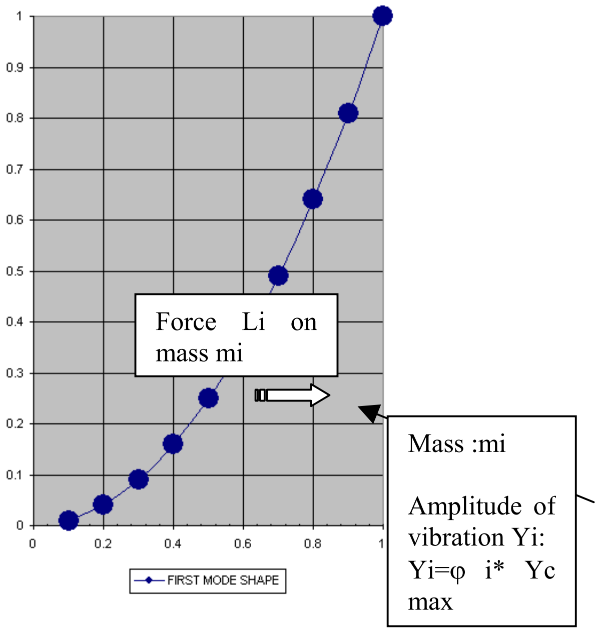


Fig 5 : simplified model of a slender structure

The structures mass is supposed to be concentrated at given location (mass mi) at elevation zi.

The inertial loads would be Li= mi * acc i
mi : individual mass mi
acc i : acceleration of the mass mi
fi : the structure frequency

acc i : $Y_i * (2 * \pi * f_i)^2$
Yi : Yc max * φ_i
φ_i : mode shape of the given mode at elevation zi.

The bending moment at stack base is:
$\Sigma\, L_i * z_i = Y_{c\,max} * \Sigma\, k*m_i*(\varphi_i)^2$

So again by reducing the amplitude Yc we also reduce the inertial force generated by the vibration.

The same principle can be adjusted for any structure and for any frequency.

2.2 PEDESTRIAN INDUCED VIBRATION

Fig 6 : general view of the Millennium footbridge in London

Fig 7 : pedestrian on the Millennium footbridge in London

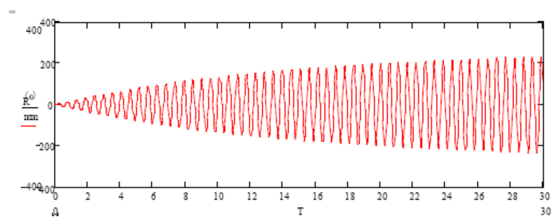


Fig 8 : resonance phenomena due to pedestrian walk on a footbridge with no damper

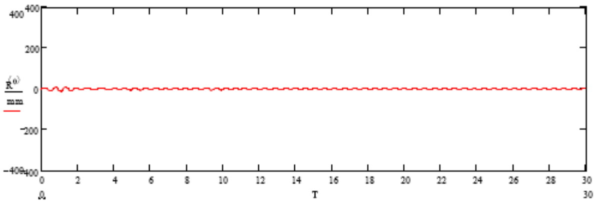


Fig 9 : resonance phenomena due to pedestrian walk on a footbridge with a damper.

Using the same formula : acc i= Yi * (2 * π * fi)^2 shows that by increasing the damping to reduce the amplitude of vibration would result in reducing the structures acceleration. This is of interest to increase the comfort of high-rise building occupants or of pedestrians on bridges, footbridges,…the sensation of comfort increases when the acceleration decreases.

3. NEW APPLICATION :ALONG WIND REDUCTION WITH A DAMPER

Some of the most recent codes have introduced the possibility of reducing the along wind by reducing the dynamic coefficient. In previous wind codes such as the French Neige et Vent 1969, the dynamic coefficient β was only a function of the type of structure (steel, reinforced concrete, pre stress concrete) and of the frequency.
In Eurocode 1 - part 2.4 wind actions on structures annexe B (wind response in wind direction) the logarithmic decrement of damping δ is introduced in the Rx coefficient (equation B.10).

On some given cases the reduction of load in the wind direction could be as great as 30% and even more resulting in huge saving in structural and foundation costs.

3.1 The theory

Eurocode 1: Basis of design and actions on structures —
Part 2-4: Actions on structures —
Wind actions

6.1 Wind forces from pressures

(1) The wind forces acting on a structure or a structural component may be determined in two ways:

- by means of global forces
- as a summation of pressures acting on surfaces provided that the structure or the structural component is not sensitive to dynamic response (c_d < 1,2, see section 9).

(2)P The global force, F_w, shall be obtained from the following expression:

$$F_w = q_{ref} \cdot c_e(z_e) \cdot c_d \cdot c_f \cdot A_{ref} \qquad (6.1)$$

where:

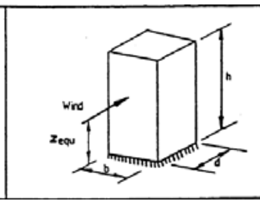
c_f force coefficient derived from section 10

A_{ref} reference area for c_f (generally the projected area of the structure normal to the wind) as defined in section 10

Vertical Structures

(e.g. buildings, stacks, towers, etc.)

$z_{equ} = 0{,}6 \cdot h \cdot z_{min}$



Annex B (informative)

Detailed procedure for in-line response

B.1. General

(1) The detailed procedure given in this annex is not appropriate for continuous bridges, cable stayed bridges and arch bridges. For such bridges specialist advice should be sought.

(2) The method for calculating the dynamic factor c_d given in this annex applies, if the following conditions are met:

- the structure corresponds to one of the standardized cases shown in Figure B.1,
- the fundamental along wind mode is uncoupled from all other modes,
- a linear elastic behaviour is applicable.

B.2 Dynamic factor

(1) The dynamic factor c_d is defined by:

$$c_d = \frac{1 + 2 \cdot g \cdot I_v(z_{equ}) \sqrt{Q_0^2 + R_x^2}}{1 + 7 \cdot I_v(z_{equ})}$$

where:

z_{equ} equivalent height of structure as given in Figure B.1

$I_v(z_{equ})$ turbulence intensity $I_v(z)$ for $z = z_{equ}$, given by equation (B.3)

g peak factor given by equ. (B.4)

Q_0 background response part given by equation (B.9)

R_x resonant response part given by equation (B.10)

Note: (1) The denominator in equation B.2 removes the simplification built into the format for c_e given in 8.4.
Thus the product $c_e \cdot c_d$ required in equation 6.1 to determine overall loads can be written as follows:

$$c_e \cdot c_d = c_r^2 \cdot c_t^2 \left(1 + 2 \cdot g \cdot I_v(z_{equ}) \sqrt{Q_0^2 + R_x^2}\right)$$

(2) The values of c_d given in section 9.3 use equation B.2 but with assumed values of wind velocity, terrain, frequency and damping, as set out in the notes in section 9.3.

B.3 Wind and structural parameters

(1) The turbulence intensity $I_v(z_{equ})$ is defined by:

$$I_v(z_{equ}) = \frac{1}{c_t(z_{equ}) \cdot \ln(z_{equ} / z_0)} \qquad \text{(B.3)}$$

Note: equation (B.3) can be written as $I_v(z_{equ}) = \frac{k_r}{c_t(z_{equ}) \cdot c_r(z_{equ})}$ using the definition of $c_r(z)$ in 8.3.

where:

$c_t(z_{equ})$ topography coefficient (see 8.4)

z_0 roughness length (see 8.2)

(2) The peak factor g is shown in Figure B.2 and defined by:

$$g = \sqrt{2 \cdot \ln(\nu t)} + \frac{0{,}6}{\sqrt{2 \cdot \ln(\nu t)}} \qquad \text{(B.4)}$$

where:

t 600 s = averaging time of the reference wind velocity, v_{ref}

ν expected frequency given by equation (B.5)

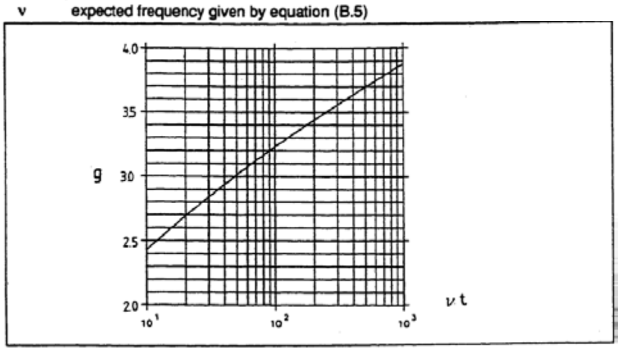


Figure B.2: Peak factor, *g*

(3) The expected frequency ν is defined by:

$$\nu = \sqrt{\frac{\nu_0^2 \cdot Q_0^2 + n_{1,x}^2 \cdot R_x^2}{Q_0^2 + R_x^2}} \qquad \text{(B.5)}$$

where:

$n_{1,x}$ fundamental frequency in [Hz] of alongwind (x) vibration of structure. Approximations for $n_{1,x}$ are given in annex C.4.

ν_0 the expected frequency in [Hz] of gust loading of rigid structures given by equation (B.6).

(4) The expected frequency of gust loading of rigid structures ν_0 is shown in Figure B.3 and is defined by:

$$\nu_0 = \frac{v_m(z_{equ})}{L_i(z_{equ})} \cdot \frac{1}{1{,}11 \cdot S^{0{,}615}} \qquad \text{(B.6)}$$

with:

$$S = 0{,}46 \cdot \left(\frac{b+h}{L_i(z_{equ})}\right) + 10{,}58 \cdot \left(\frac{\sqrt{b \cdot h}}{L_i(z_{equ})}\right) \qquad \text{(B.7)}$$

where:

b, h width, height of structure as given in Figure B.1

$v_m(z_{equ})$ mean wind velocity $v_m(z)$ for $z = z_{equ}$ given by equation (8.1).

$L_i(z_{equ})$ integral length scale of turbulence for $z = z_{equ}$ given by equation (B.8)

(5) The integral length scale of turbulence $L_i(z)$ is shown in Figure B.4 and is defined by:

$L_i(z) = 300 \cdot (z/300)^{\varepsilon}$	(L_i, z in m)	for $z_{min} \le z \le 300$ m	(B.8)
$L_i(z) = 300 \cdot (z_{min}/300)^{\varepsilon}$	(L_i, z in m)	for $z \le z_{min}$	
$L_i(z) = 300$ m		for $z > 300$ m	

where:

ε, z_{min} are given in Table 8.1

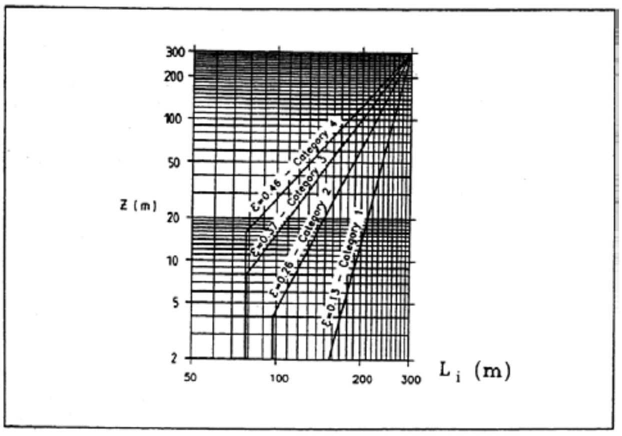


Figure B.4: Integral length scale of turbulence, $L_i(z)$

(6) The background response part Q_0 is shown in Figure B.5 and is defined by:

$$Q_0^2 = \frac{1}{1 + 0{,}9 \cdot \left(\frac{b+h}{L_i(z_{equ})}\right)^{0{,}63}} \qquad \text{(B.9)}$$

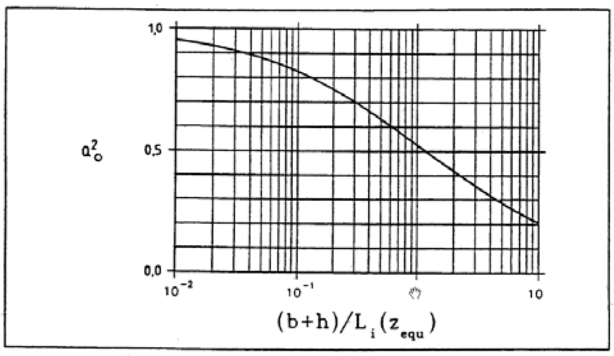


Figure B.5: Background response part Q_0

(7) The resonant response part R_x is defined by:

$$R_x^2 = \frac{\pi^2}{2 \cdot \delta} \cdot R_N \cdot R_h \cdot R_b \qquad \text{(B.10)}$$

damping

where:

δ logarithmic damping decrement of alongwind vibration. Standard values for δ are given in C.4

R_N nondimensional power spectral density function given by equation (B.11.)

R_h, R_b, aerodynamic admittance functions given by equation (B.12)

(8) The resonant nondimensional power spectral density function R_N is shown in Figure B.6 and is defined by:

$$R_N = \frac{n_{1,x} \cdot S_v(n_x)}{\sigma_v^2} = \frac{6{,}8 \cdot N_x}{(1 + 10{,}2 \cdot N_x)^{5/3}} \qquad \text{(B.11)}$$

with:

$$N_x = \frac{n_{1,x} \cdot L_i(z_{equ})}{v_m(z_{equ})} \qquad \text{(B.12)}$$

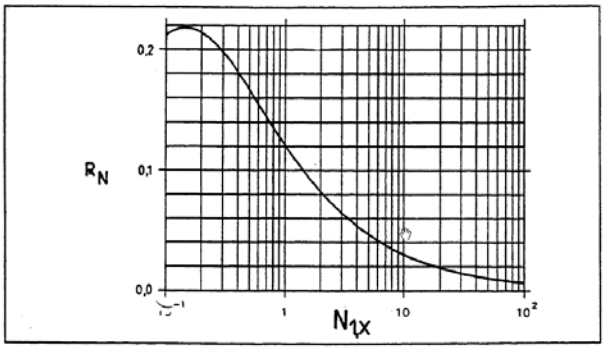


Figure B.6: Nondimensional power spectral density frequency function R_N

(9) The aerodynamic admittance functions R_h and R_b for uniform displacement (fundamental mode shape without node point) are expressed in terms of the function

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}\cdot(1-e^{-2\eta}) \quad \text{for } \eta > 0$$

$$R_\ell = 1 \quad \text{for } \eta = 0 \qquad \text{(B.13)}$$

with:

$$R_h = R_\ell \quad \text{setting } \eta = \frac{4{,}6\cdot N_{1,x}\cdot h}{L_i(z_{equ})} \qquad \text{(B.14)}$$

$$R_b = R_\ell \quad \text{setting } \eta = \frac{4{,}6\cdot N_{1,x}\cdot b}{L_i(z_{equ})} \qquad \text{(B.15)}$$

For mode shapes with internal node points more detailed calculations shall be used.

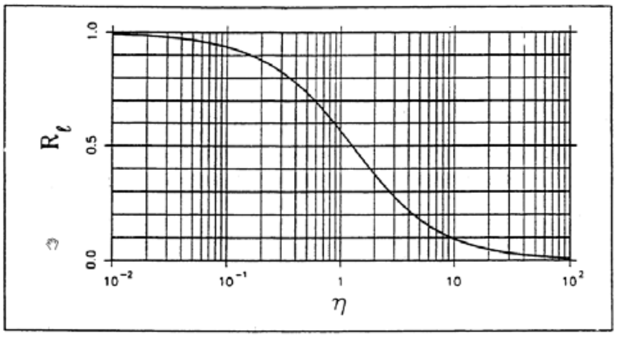


Figure B.7: Aerodynamic admittance function R_ℓ (ℓ = h, b)

(9) The aerodynamic admittance functions R_h and R_b for uniform displacement (fundamental mode shape without node point) are expressed in terms of the function

$$R_\ell = \frac{1}{\eta} - \frac{1}{2\eta^2}\cdot(1-e^{-2\eta}) \quad \text{for } \eta > 0$$

$$R_\ell = 1 \quad \text{for } \eta = 0 \qquad \text{(B.13)}$$

with:

$$R_h = R_\ell \quad \text{setting } \eta = \frac{4{,}6\cdot N_{1,x}\cdot h}{L_i(z_{equ})} \qquad \text{(B.14)}$$

$$R_b = R_\ell \quad \text{setting } \eta = \frac{4{,}6\cdot N_{1,x}\cdot b}{L_i(z_{equ})} \qquad \text{(B.15)}$$

For mode shapes with internal node points more detailed calculations shall be used.

Conclusion : the total along wind force is affected by the dynamical factor Cd. Many parameters are taken into consideration in calculation of the Cd factor . The main parameters are :

- the gust factor g depending of structure size (diameter b and height h).(g is a function of ν depending of νo depending of S which is a function of b and h..νo is also a function of Vm: mean wind velocity at z equi level.

- the soil rugosity (cf Li : integral length of scale of turbulence).

- the structure height affecting different others parameters such as Li.

- δ : the logarithmic damping decrement of the structure..

What can we do to reduce the along wind load dynamical coefficient cd?

The stack location is given by the client so the soil rugosity can not be changed.

The stack height h and the top diameter at least are specified by the client of code for emission or draft consideration. Very little modification could be done.

Vm (mean wind speed) depend of stack height, soil rugosity , topography . Cannot be changed.

As a result only changing the δ log decrement could have a direct action on the Cd dynamical coef

3.2 Case study on steels stacks -calculation of the reduction of cd with different value of δ

Four typical stacks of 30 m ,40 m, 60 m and 100 m are given as examples.

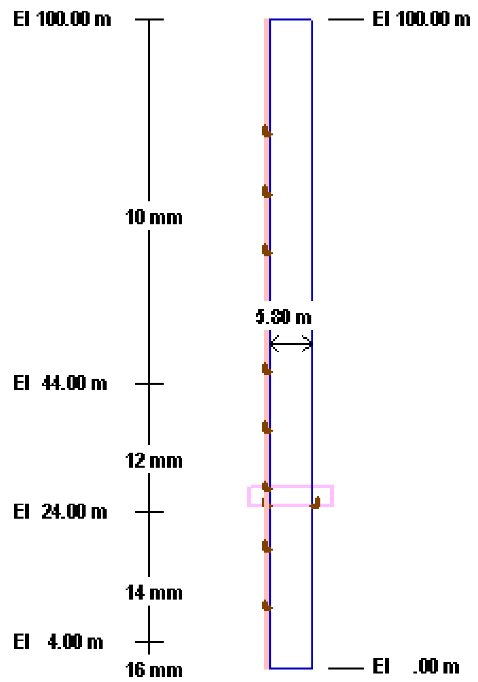


Fig 10 : sketch of a 100 m steel stack under construction in Russia

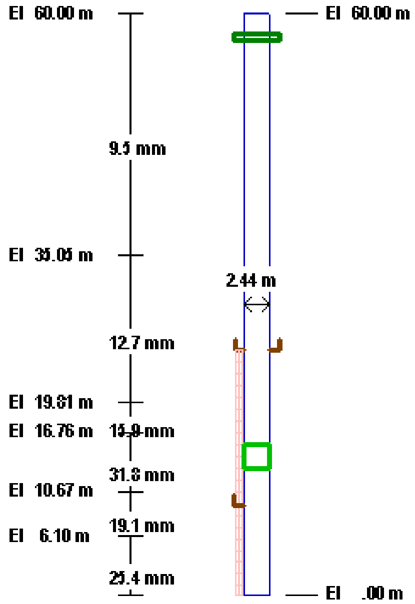


Fig 11 : sketch of a 60 m steel stack

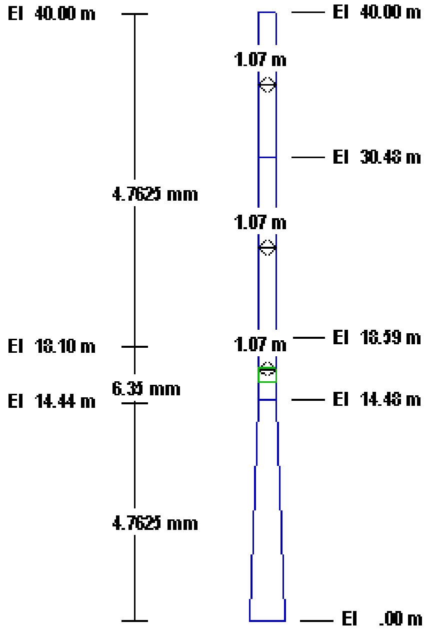


Fig 12 : sketch of a 40 m steel stack.

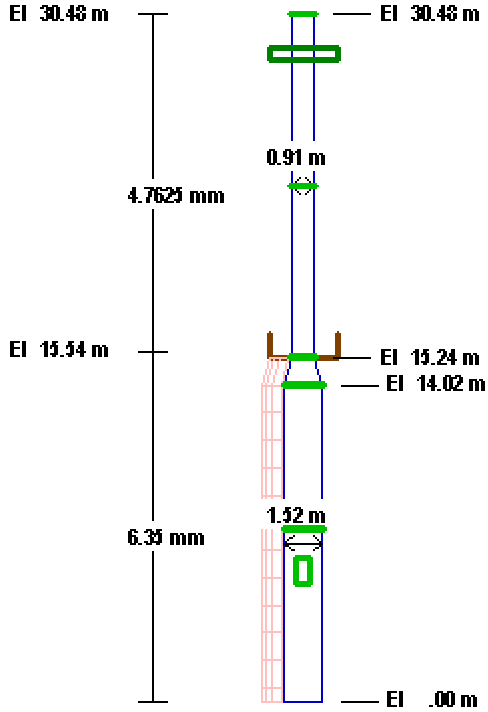


Fig 13 : sketch of a 30 m steel stack

For each stack we have studied three different location in order to see the influence of the soil rugosity on the results.

Terrain category 1 : Rough open sea, lakes with at least 5 km fetch upwind and smooth flat country without obstacles.

Terrain category 2 : Farmland with boundary hedge, occasional small farm structures, houses or trees

Terrain category 5 :urban area with average building above 15 m

Value of the dynamical coef as a function of log damping dor different stacks	Log damping	TERRAIN CATEGORIES		
		1	2	5
	Li=	260	233	180
Stack 100 m	0.015	1.959	2.067	2.103
Diameter : 5.80 m	0.050	1.385	1.422	1.402
first mode frequency : 0.63 Hz	0.100	1.196	1.210	1.175
	0.150	1.118	1.123	1.082
	Li=	230	210	140
Stack 60 m	0.015	2.003	2.120	2.153
Diameter : 2.44 m	0.050	1.410	1.451	1.427
first mode frequency : 0.77 Hz	0.100	1.217	1.239	1.193
	0.150	1.137	1.145	1.099
	Li=	233	190	110
Stack 40 m	0.015	2.035	2.175	2.208
Diameter : 1.07 m	0.050	1.430	1.482	1.453
first mode frequency : 0.91 Hz	0.100	1.233	1.257	1.212
	0.150	1.153	1.165	1.115
	Li=	225	175	95
Stack 30 m	0.015	1.727	1.804	1.728
Diameter : 0.91 m	0.050	1.287	1.309	1.236
first mode frequency : 1.81 Hz	0.100	1.151	1.157	1.089
	0.150	1.097	1.097	1.031

Table 1 : summary of the variation of the dynamical coefficient Cd for different types o stacks with different log damping.

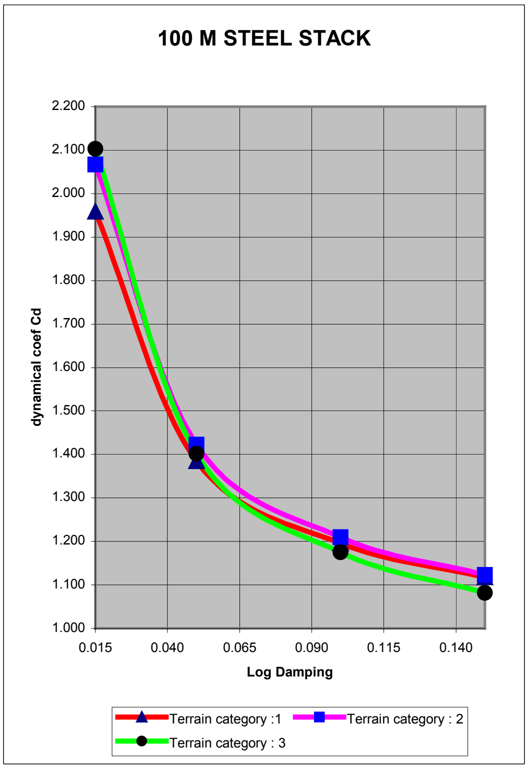


Fig 13 : Variation of the dynamical coefficient for a 100 m stack versus log damping for different Terrain rugosity category

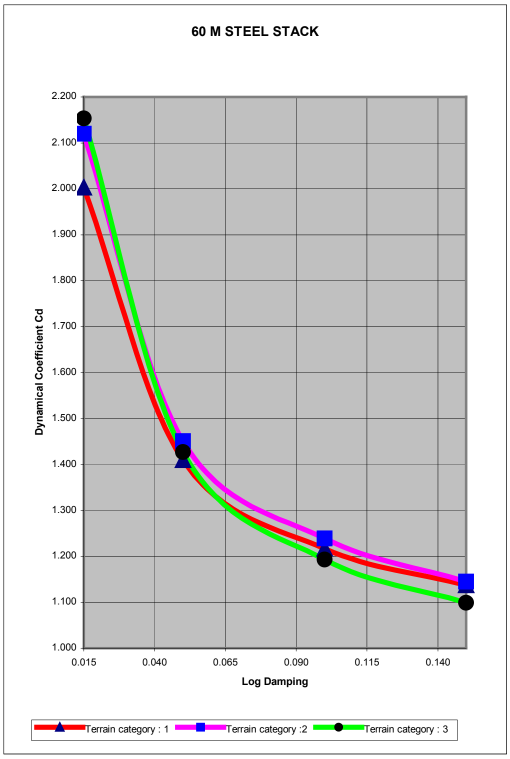


Fig 14 : Variation of the dynamical coefficient for a 60 m stack versus log damping for different Terrain rugosity category

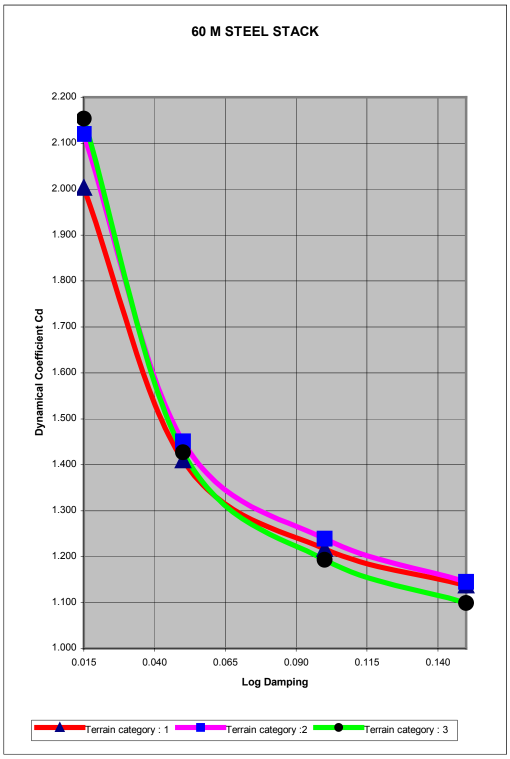


Fig 15 : Variation of the dynamical coefficient for a 40 m stack versus log damping for different Terrain rugosity category

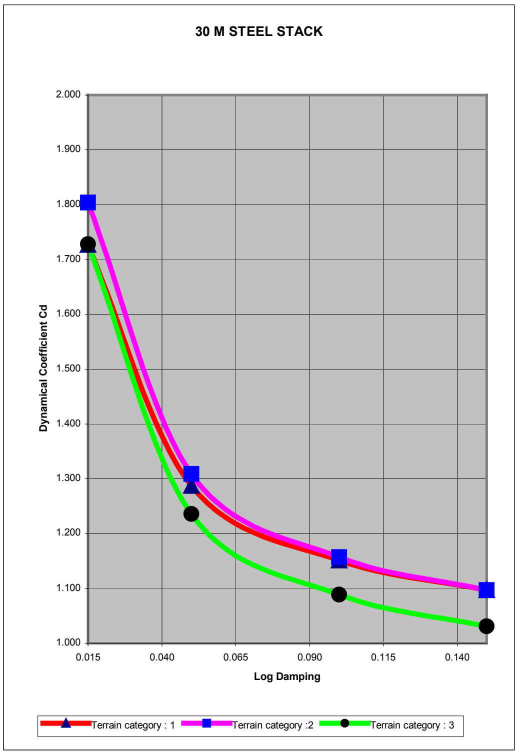


Fig 15 : Variation of the dynamical coefficient for a 30 m stack versus log damping for different Terrain rugosity category

From the above curve based on a sample of stacks with height ranging from 30 m up to 100 m with three terrain category we can see that the dynamical coefficient is decreasing the same way while the log damping is increasing.

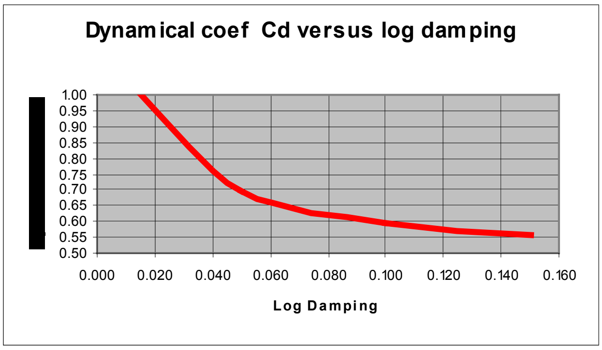


Fig 16 Variation of the dynamical Coef Cd versus the log damping

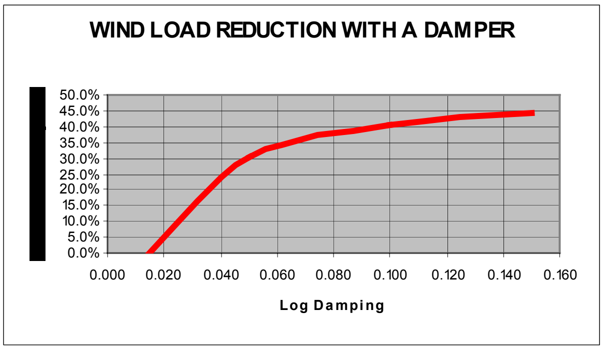


Fig 17 Average wind loads reduction by using a damper.

3.3 Conclusion

Using a vibration damper can help reducing the Cd dynamical coefficient by huge percentage. As the wind load is Fw=Q ref* Ce*Cd*Ct*A ref then the same reduction is expected on the total wind force applied onto the stack.

Many stacks have a structural log damping smaller than 0.20. Adding a damping system with giving to the stack a total log damping of 0.100 give a along wind loading reduction of 40-0.05=35%.

3.4 Case study refurbishment of a 140 m concrete stack by using a damper

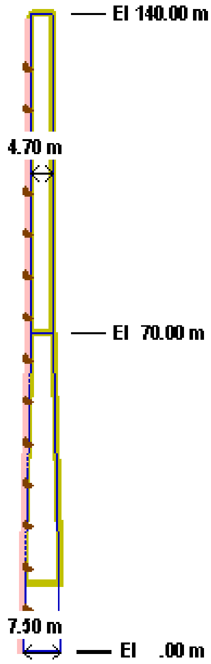


Fig 18 :general view of the concrete

This stack was built long time ago and show some area with large defect mainly cracks and poor quality concrete.

Investigation was carried out and show that the general behavior of the stack was not really the expected one. The measured frequency was more smaller than the calculated one. Explanation of this difference could not be the be resulting of foundation weakness because this one was on pile with a strong concrete slab. The Young Modulus of the concrete measured on cylindrical core was between 22 500 Mpa and 25 000 Mpa much smaller than normal value. The concrete strength under compression was above 30 Mpa which was acceptable with a max compressive stress of 21 Mpa. The measured yield stress on steel reinforcement was nearly 450 Mpa on most of the samples.

Having recalculated the stack with the new wind condition for this site we state than t a huge ovrstressing of the vertical reinforcement was expected : between the stack bottom and the level 85 m the stresses in the vertical reinforcement was more than 30% at atsome level the max stresses were above the breaking stress in the reinforcement.

<u>Two solutions were recommended :</u>

a) to make a new concrete shaft around the existing one between ground level and level 90 m

b) to place a vibration at the stack top.

The new concrete shaft to be heavily reinforced so that to have acceptable stress in the vertical reinforcement. The additional dead weight to e also acceptable for the foundation block and pile.