SIGN BRIDGES : IMPROVEMENT OF THEIR WIND RESISTANCE

J. BERTHELLEMY
Engineering Structures Techniques Centre (CTOA), Sétra, France.
direction.setra@equipement.gouv.fr

ABSTRACT

Sign bridges underwent important damage at the time of the storms which crossed France in January 1998, then especially in December 1999. Old structures with already weak points, like fatigue cracks, were naturally eliminated on these occasions.
The failure of a sign bridge structure can have serious consequences for the road users, because the object can still cause accidents after its fall.
In parallel, the density of the road network converges, so that the desirable service lifespan of a sign bridge structure passed, in one decade, from 15 years to approximately 30 years. It was thus decided to upgrade the French standard [11] with the aim of reinforcing the new sign bridge structures.
The introduction of the eurocodes makes it possible to complement this evolution in particular by introducing the justification of the structures to resist the fatigue caused by the wind.
The maintenance and the monitoring of existing sign bridges are strong concerns for road managers. These topics were the subject of the publication of a technical guide detailing the inspections of the structures to be carried out. [7]
Lastly, experimental sign bridges projects are engaged with the objective to carry out some robust structures, in particular with redundant columns.

1. PROBLEMS RAISED BY THE MAINTENANCE OF SIGN BRIDGES.

Sign bridges, Gantries, Cantilevers and High Masts, can reach important dimensions: several tens of metres for gantries, fifteen metres of span for cantilevers. Their calculation is mainly governed by the effects of the wind. They often carry ordinary signs, and sometimes the heavier equipment of variable message signs (VMS).

The number of French sign bridges stations is estimated today at approximately 15.000 units. On the National owned Highways, the sign bridges are not regarded as structures, but as road equipments, and they are maintained on the road budget.
The system of the guarantees also differs from that of the structures. There is not a decennial guarantee, because of absence of jurisprudence, but only one decennial responsibility for the manufacturer. Although is not with this type of text to discuss this question, the French Standard XP P 98-550 [11] limits in a purely informative part, the responsibility of the manufacturer to guarantee a one single year only, except if maintenance is carried out according to very demanding criteria.
However, the management of sign bridges raises the same technical questions as that of structures. They enter within the framework of the following general problems:

- Which is my inheritance ?
- In which conditions is it ?
- How to make the number of my structures durable ?
- How much will it costs ?

The census and the systematic evaluation of the sign bridges are in phase of generalization. But a system of collection of all the data for these structures does not yet exist in France. From the inventory of the observed pathology comes out a check-list of 71 different types of degradations, that can affect all parts of the sign bridges, at various frequencies and with more or less high degrees of gravity.

In addition, the rhythm of the construction of new roads is slowing down because of the maturity and of the density of the National Highway network. The new roads and the transformations of highway interchanges are rare, so that it is estimated that the functional lifespan to be hoped for a sign bridge would have passed in one decade from 15 years, to approximately 30 years.

Consequently, economic and legal risks induced by an insufficient maintenance can be very important. The fall of a sign bridge cannot certainly be compared with the collapse of a bridge, because in the case of the sign bridge, it is easy to unblock the jammed road and to restore circulation. However for example, the financial incidence that the fall of a gantry would have on an interurban highway, by integrating in particular the cost of the victims, is estimated at 12 times the value of replacement of the work with a new one. In addition, it is without counting the immediate echo that such an event would find, through the users, the media, and also the penal judge. Because the user-citizen accepts less and less the fate to be evoked, he seeks after a person in charge.

Naturally, the road manager, whose negligence can be at the origin of the disaster, will at first be blamed. So the bridge owners take in time radical safety measures, i.e. often the disassembling of the structures. Thus we fortunately do not have sufficient statistics on the accidents, and we are not able to carry out an analysis of risks. One considers on the other hand the lifespan real average of the structures at approximately twenty years, which is probably far away from the economic optimum.

Because of these concerns, a study of the DDE of the Rhone introduced for the first time for sign bridges, in the services of the French Ministry of the Transport, the definition of a method of maintenance, which classify the observed degradations according to a hierarchical system in order to prioritize the interventions [3].

Today, whereas the durability of sign bridges (Gantries, Cantilevers and High Masts), is a strong concern for road managers, the definition of a clear policy of the structures Owners, and in particular of the Administration is imperative in the field of specifications for construction and management of these works. It is the reason of their greater implication in the revision of the French standard XP P 98-550 whose drafting does not only concern the manufacturers. The objective is that these structures take part of sustainable development.

During the last five years, Sétra played a leading role in the evolution of these specifications, and was associated to the drafting of the technical guide published by LCPC : "Detailed, initial and periodic inspections of Sign Bridges". [7] This technical guide suggests precaution measures to take, adapted to every case of structural degradation.

2. SHOCKS EXPOSURE : CONCEPTS OF HEADROOM AND OF GAUGE

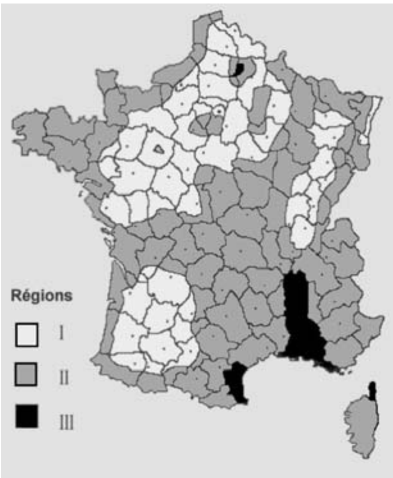
The concept of height "gauge” which appeared in old standard XP P 98-550 relates to the user's vehicle. This term was employed by error in the place of the term “headroom” to indicate the free room under the signs of the sign bridge. In fact, the headroom is the sum of the gauge plus a safety revenge of 500 millimetres, minimal value for the French

National Highway roads. The usual total headroom advised in the new standard under the signs is 5,500 meters.

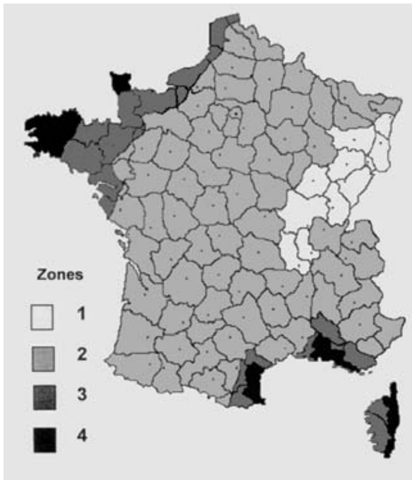
Eurocode 1 will soon add an additional condition to the sign bridges design because it envisages lawful shocks to which the structures built above the road must resist. Non-fusible carrying structures, i.e. the transverse beams, are concerned and should be dimensioned to resist shocks up to six meters height above the roadway.
This specification leads in practice to place the low point of the transverse beam at six meters height above the roadway. This thus leads in many cases to a better centring of the transverse beam compared to the signs, which are fusible elements whose low point can remain with 5,500 meters height above the roadway. That will thus prevent from placing the transverse beam as low as possible compared to the signs, and will limit consequently the distortion warping stresses in the transverse beam, which are not always well taken into account.

3. MODIFICATION OF THE WINDS MAP FOR SIGN BRIDGES

Modifying text number 2 of the map of the zones of wind for the French Standard is appeared in December 1999. It is for example available at the CSTB (book 3182, delivery 405).
This text comprises national maps in appendix. The map established for France takes account of more recent weather data, and is much more complete than in 1965.



Winds map of 1965



Winds map of 1999

The new revised standard XP P 98-550-1 will appear soon will be based on the new map. The weather events of the end of 1999 have confirmed with force the relevance of the modifications made to the map, which presents a rather different aspect from this which was given before.

4. ADOPTION OF AN ADAPTED DRAG COEFFICIENT

The application of Eurocodes to sign bridges is still in debate. However, for the structures owners which are responsible for this choice, the application of Eurocodes allows to markedly improve the reliability of sign bridges compared to the current standards.
Thus, current standard XP P 98-550 leaves the choice for the action of the wind between the application of NV65 rules, and the use of “package” values. The same drag coefficient

for a rectangular VMS box, with length double than height, can thus be taken as well equal to **1,35** according to NV65 rules or equal to **1,75** with the “package” values !

New revised standard XP P 98-550-1 will avoid this ambiguity by proposing only minimal values, obtained with a single drag coefficient of **1,75** independent of the shape of the rectangular sign, and in very close conformity with Eurocode 1 [5].

5. INTRODUCTION OF A VERTICAL WIND

Vertical loads were introduced to supplement standard XP P 98-550. The pressures exerted by the wind in the vertical direction account for **30%** of the pressures exerted in the horizontal direction.

The components vertical and horizontal are concomitant, the vertical component takes account of the aerodynamic effects and supplements the effect of the wind described in standard XP P 98-550. In accordance with the proposal of the CSTB, where sign bridges were studied out in wind tunnel tests, the objective is to sufficiently rigidify in all the cases the structures, to avoid being in fields where aero-elastic instabilities are to be feared. The galloping, also known as Den Hartog instability, would in particular be likely to appear with the too flexible structures carrying small dimension signs, and thus presenting a weak aerodynamic damping. Galloping must imperatively be avoided.

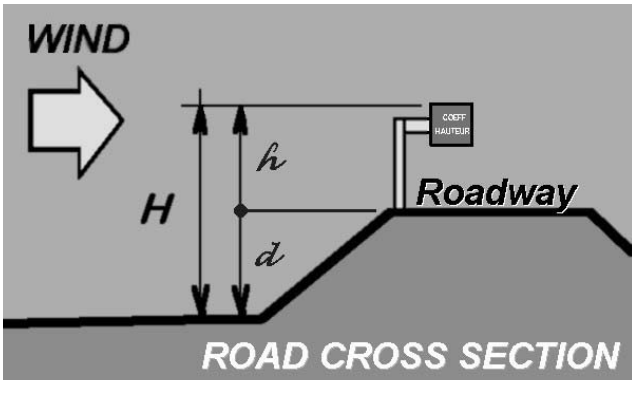
The orientations of the horizontal loads to take into account are : of face and back, of left and right side. The orientations of the vertical loads to take into account are: from top to bottom, and from bottom to top.

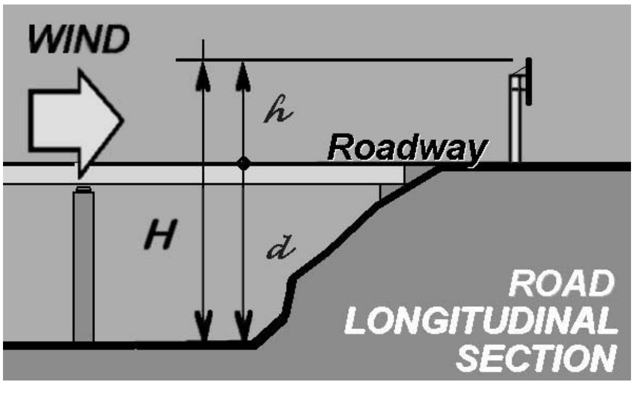
The action of the wind is exerted as well on the signs and their fixings as on the carrying structure. Of course, the surfaces exposed to the horizontal wind are much more important than those exposed to the vertical wind.

6. RATIONALIZATION OF THE HEIGHT COEFFICIENT

Concerning the height coefficient, the reading of old standard XP P 98-550 could lend to confusion.

It is therefore specified in the new revised standard XP P 98-550-1 that the height coefficient, taken into account by the contractor in his offer and his calculations is not based on the height h above the roadway, but on the total height **H=h+d** above the aerodynamic environment, as for example the figures which follow, specify it :





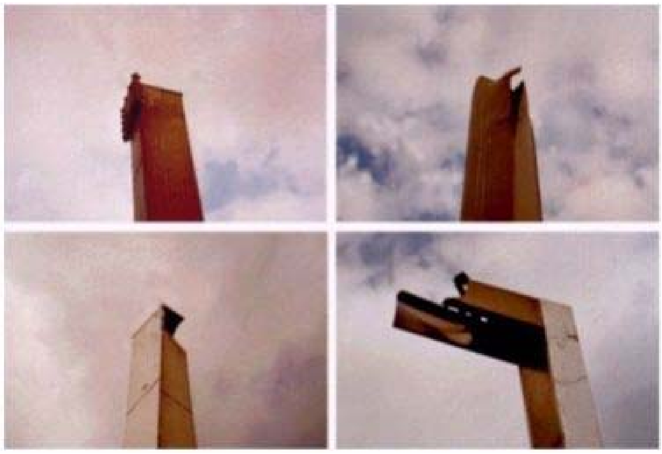
During the development of his proposition to answer the call for tender, the contractor is supposed to have taken knowledge of the places on which the structures are to be built. In the exceptional cases where the height H is higher than 10m, the values characteristic of the wind are multiplied by the following height coefficient :

$$2{,}5 * (H+18)/(H+60)$$

7. DESIGN RULES FOR FATIGUE

Eurocodes bring other scientific answers and adapted models of calculation. Thus for example they take into account the phenomenon of metal fatigue, for aluminium or steel. Sign bridges design is mainly governed by wind, action eminently variable, which severely affects the structures in fatigue. [10]. After each major storm, there are structures of sign bridges which are made unsuitable for any use. Their demolition is then decided without calling neither upon the manufacturer to repair, nor at a laboratory to analyze the exact nature of the initial fissures.

Storm of January 1998
(*Photo J. Berthellemy*)



Storms of December 1999
(*Photographs of DDE*)

The fatigue checking of structures sign bridges is not approached in standard XP P 98-550 and will not be it in next revised standard XP P 98-550-1. This subject will be covered only in one future part 2 of this standard. Eurocodes however propose a step which makes it possible to apprehend the question of the fatigue life of the structures, and to treat this question in the drafting of the call for tender.

It thus appeared to us interesting to propose in the publication [4] this methodology for the checking of the sign bridges in regard to fatigue under the effects of the wind perpendicular to the plan of the structure.

7.1. Implementation of the principles of Eurocode 1

(appendix B of the standard In-1991-1-4-actions of the wind of January 2005)

The checking is based on the action of fatigue only caused by the turbulence of the horizontal wind. It supposes that the structure does not have oscillations and that safety with respect to the aero-elastic phenomena suitable for the structure is improved in addition, in particular thanks to the introduction of a vertical static wind.

Eurocode gives in its appendix B the number of loadings to be considered for a dynamic response. These data make it possible to lay down the rule suggested below, whose justification is detailed further.

Fatigue due to the wind is modelled by the action of a “wind of fatigue”, equal to a fraction of that of the characteristic wind, which is the wind with a return period of 50 years, corresponding to the usual SLS (Serviceability Limit State).

- **For a fatigue life of 25 years, the action of the wind of fatigue will be taken equal to 22% of that of the characteristic wind;**
- **For a fatigue life of 50 years, the action of the wind of fatigue will be taken equal to 26% of that of the characteristic wind.**

Calculation consists in checking that the structure resists 2 million cycles of the wind of fatigue, i.e. that the stresses due to the wind of fatigue are lower or equal to the categories of detail of the joints.

According to principles of Eurocodes, it is appropriate to retain a safety coefficient in fatigue γ_{MF} depending on the accessibility of the justified element, on the frequency of the inspections and on the consequences of a failure.

Table 3.1: Partial safety factor for fatigue strength γ_{Mf}

Safety concept	Consequence of failure	
	Low consequence	High consequence
Damage tolerant concept	1,00	1,15
Safe life concept	1,15	1,35

For the structures being the object of a periodic monitoring, in accordance with the indications given in the appendix C of standard XP P 98-550, and with those of the LCPC guide for the Inspection of the sign bridges [7] it seems possible to retain a coefficient γ_{MF} equal to **1,15**.

The characteristic pressure of the wind noted **Qkv** is defined in experimental standard XP P 98-550. Its value was re-examined in version XP P 98-550-1. It depends on the zone of wind, the new zones of wind having already been the subject of a publication in the Structures Bulletin of Sétra n°36 [9].

Zone of wind	Characteristic wind pressure (Pa)	Wind of fatigue **25 years** duration Pressure (Pa)	Wind of fatigue **50 years** duration Pressure (Pa)
1	1800	396	468
2	2200	484	572
3	2700	594	702
4	3200	704	832
5	4300	946	1118

7.2. Justification of the proposed method

One considers an assemblage in which the characteristic wind creates a stress of 100 MPa. It is supposed that this assemblage has a category of detail of known value X.

By Miner summation, one can calculate the damage in fatigue it at the end of the wished durability-span, by considering the histogram of the goings beyond of level defined in the B3 paragraph of the appendix B of EN 1991-1-4. This histogram indeed makes it possible to establish the spectrum of the actions of the wind. In this calculation the shape of the diagram of Wöhler (SN curve) intervenes. One considers a SN curve with two slopes and with truncation.

The first slope is – $1/m_1$, second is – $1/m_2$:

- $N < 5.\ 10^6$: the slope is - $1/m_1$
- $5.\ 10^6 < N < 10^8$: the slope is – $1/m_2$
- $10^8 < N$ null slope (truncation)

For all the aluminium alloy joints, as for steel, one has $m_2 = m_1 + 2$.
Calculations which follow are made with this value of m_2.
By successive iterations, one determines the value of X which leads to a damage equal to 1 at the end of the desired durability -span.

If the wind of fatigue creates in the assemblage a stress variation equal to X, then the assemblage will resist exactly 2 million cycles of wind of fatigue. Thus X must be the action of the wind of fatigue on this assemblage. However the action of the characteristic wind is 100. Thus the action of the fatigue wind is X/100 of that the characteristic wind.

For a service lifespan of 25 years, the value of the category of detail leading to a damage equal to 1, under the conventional spectrum of fatigue, when the characteristic wind gives a stress equal to 100, is indicated in the table below.

First slope of SN curve (m_1)	Value fixed for X (category of detail)
3	22,6
3,5	22
4	21,6
4,5	21,5
5	21,8
6	22,6

The value of X varies from 21,5 to 22,6% of the stress under the characteristic wind.
This result justifies the proposal to retain, for one fatigue lifespan of 25 years, an action of the fatigue wind equal to 22% of that of the characteristic wind.

For one service lifespan 50 years, the value of the category of detail leading to a damage equal to 1, under the conventional spectrum of fatigue, when the characteristic wind gives a stress equal to 100, is indicated in the table below.

First slope of SN curve (m_1)	Value fixed for X (category of detail)
3	26,7
3,5	25,5
4	24,9
4,5	24,6
5	24,6
6	25,2

This result justifies the proposal to retain, for one fatigue lifespan of 50 years, an action of the fatigue wind equal to 26% of that of the characteristic wind.

7.3. Example of application to anchor-rods

A justification with respect to fatigue can be produced for:

- anchor-rods and bolted joints,
- welded joints, by distinguishing full penetration welded joints and weld toes, and by referring to Eurocode 3 for steel and Eurocode 9 for aluminium alloys.
- Signs fastening systems.

The level of the fatigue loads is related to the service lifespan wished by the customer. In order to simplify, we propose two classes: service lifespan of twenty five years for **ordinary works,** and of fifty years for **robust works**.

But according to the needs, a call for tender could easily require a number of years of service T intermediate between 25 and 50, or higher than 50 according to the desired durability, and specify the corresponding fatigue wind. By simplification, one can estimate in term of stresses the effect of the wind of fatigue for one lifespan of T years at:

$$\Delta\sigma_{T\_years} = (T/50)^{1/4,14} * 0,26 * \sigma_{Wind\ characteristic}$$

One checks simply

$$\Delta\sigma_{T\_years} * \gamma_{MF} < \Delta\sigma_{2Mcycles\ Class}$$

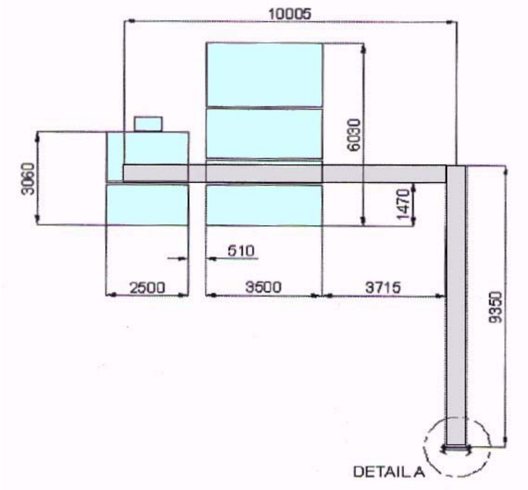
Let us take as example anchor-rods out of rolled steel bars, with yield stress of 355 MPa. In this case, the class of fatigue is:

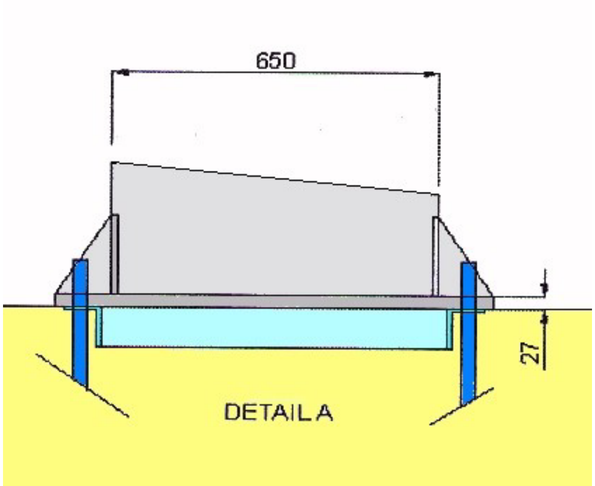
$$\Delta\sigma_{2Mcycles\ Class} = 50\ MPa$$

under the conditions of Eurocode 3 (pr_EN) or standard XP 22-311. Let us note that the EN project envisages an effect of dimension for higher than 30 mm diameters :

$$\Delta\sigma_{2Mcycles\ Class} = (30/\Phi)^{0.25} * 50\ MPa$$

For the simplicity of the example, one will choose a base assemblage where the base-plate is reinforced by a framework of angles, but a similar reasoning can be carried out on other details.





In this case, the dimensioning of the anchor-rods in the concrete is currently led to the ULS (ultimate limit state) of resistance by making sure that one remains in the elastic range for all the anchor-rods.

Each calculation being an elastic design, one passes in fact linearly of the ULS calculation of resistance to the fatigue analysis. There is no secondary bending moment in the anchor-rods since shear and torsion are resisted by the framework of angles.

In this precise case of steel anchor-rods not subjected to secondary bending moments, the comparison is immediate and the lifespan of characteristic D_service fatigue (calculated with a coefficient safety γ_{MF} equal to **1,00)** can be easily given for rods with 355 MPa of yield stress and classified at 50 MPa in fatigue: it is **22 years.**

D_service = 50 * ((50 * 1,50)/(1,00 * 355 * 0.26)) ** 4,14 = **22 years**

This result confirms that the current structures, dimensioned according to standard XP P 98-550, must be the subject of a monitoring, like annexes it C informative invites the building owners to do it.
Older structures are likely to present starters of fatigue cracks in net bottom of the screw cuttings, for the most loaded rod, generally placed in the corner of a rectangular base.

The structures which would have been designed in resistance with rods of higher yield stress have a service lifespan even more reduced. Rods of more than 355 MPa of yield stress are to be avoided, because their classification in fatigue remains the same one.

On the other hand anchor-rods of the Sixties still out of S240 steel of 235 MPa of yield stress will undoubtedly have a longer service lifespan, provided corrosion does not weaken the anchor-rods:

D_service = 50 * ((50 * 1,50)/(1,00 * 235 * 0.26)) ** 4,14 = **52 years**

A method of detection of cracks in the anchor-rods, founded on the principle of US (reflection of the ultrasounds), was developed in laboratory [6]. It is from now on possible to apply it in situ because it gives reliable results, with the proviso of taking into account its operational as well as physical limits in particular with respect to the minimal diameter of the rods. Uncertainty remains for small diameter rods. In case of doubt, the disassembling of the bolts, and even of the all structure, remains necessary to allow a thorough investigation of the anchor-rods, which can then be tested mechanically one by one. For candelabra, mechanical tests can on the other hand usually be practiced without disassembling [8].

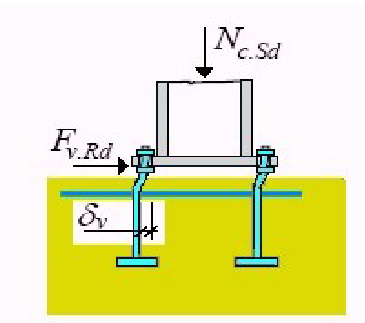
7.4. Particular case of the open assemblage

The open assemblage is today the most current adopted solution, because of its simplicity of implementation. Nevertheless, it supposes that particular provisions are taken:

- Anti-corrosion protection of the anchor-rods must have a service lifespan compatible with the lifespan awaited of the structure. In the absence of sacrificial extra thickness, this provision limits its employment to not very chemically aggressive environments, if one waits for a service lifespan of the structure higher than 15 years;
- The foundation block must slightly be raised and designed to avoid any water stagnation between base plate and concrete;
- Anchor-rods must be correctly ventilated and protected from water projections coming from the roadway.

Above defined protections are necessary, because the proximity of the road often constitutes a chemically very aggressive environment, as it was established by the measurements reported in [12] : a real size experimentation in the median strip of the A1 highway highlighted a speed of sacrificial consumption of the zinc reaching between 2,5 μm/year and 4,4 μm/year. For a thickness of the by galvanization deposited zinc of 80 μm, it results from this study a service lifespan of about 15 years near a strongly salted highway, whereas one can hope a 50 years durability in open country.

In the case of an open assemblage, the fatigue analysis must take account of the local bending of the rods under the horizontal force, by regarding the rods as connected without hinge on the level of the reinforcement bars of the concrete foundation block on the one hand, and on the lower level of the nut under base plate on the other hand, if the nut is well tightened.



Open assemblage:
Local effect in the anchor-rods.

Possible alternative to the open assemblage is the use of a standardized propping product which is cast in place under the base plate. This work requires a very particular care then, otherwise the corrosion of the anchor-rods can develop and remain dissimulated by the propping mortar. This type of assemblage must thus be reserved for the cases of the strongly aggressive exposures (salt spray, projections of de-icing salt).
With a filling of propping product, carried out according to the code of practice, controlled, and maintained during the life of the work, it is acceptable to not cumulate in fatigue the effect of the local bending moments in the rods with the other effects.

8 IMPLEMENTATION OF THE SPECIFICATIONS OF THIS ARTICLE

Sign bridge structures are to calculate in France on a case-by-case basis as they are subjected to different wind loads according to their location. But in the past, the technical specifications of the sign bridge structures, coming with the call for tender were often incomplete, which could lead for example the Contractor to make in the place of the Owner, the choice of the site and drag coefficients.

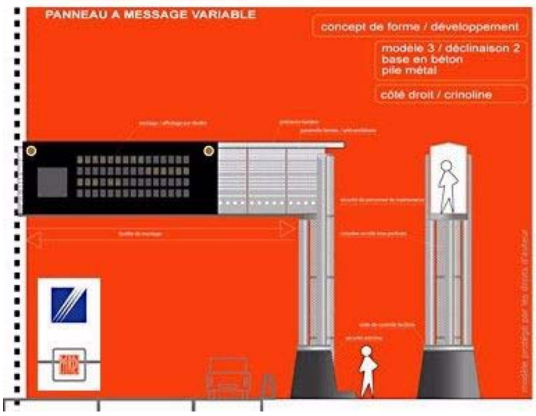
But on the other hand, recommendations for the redaction of new sign bridges contracts, in order to obtain the necessary quality for durable works are available today: since 2003, Sétra provides in the form of experimental technical specifications and clauses, the help for the Owners who need the robustness of the ordered products. These documents can already be used while waiting for the publication by AFNOR of standard XP P 98-550-1.

The experimental technical specifications and clauses of Sétra takes into account the regulation French books called CCTG (general technical specifications for the contracts of the State concerning the works of civil engineering), without calling into question the usual requirements of the certification by ASQUER (Association for the qualification of the road equipments) which were made compulsory in France by an "Arrêté".

The provided technical specifications and clauses also define the development process of a sign bridge, since the period of preparation until the reception of the work, in form of the necessary requirements of a quality insurance plan. Thus, to each stage, corresponds a point of stop, for the validation of the design by the Owner. Controls are then obliged passages, and arrival of realisation defects, such as we know otherwise in some cases, is very reduced.

8.1 *Application to sign bridges*

The study for the area of Saint-Etienne of architectural structures carrying VMS constitutes one of the first applications of the precedent concepts, on the initiative of the DDE Loire. (former local Direction of the French Transport Ministry). The building Owner is the State, and the design is ensured by Sétra, associated to the *"Pardi-Design"* office which mission is the aesthetics of the project.



It is to be noted that the concrete block of foundation rises until the height of the eyes, in order to put the interface and its anchor-rods at the cover of projections. The interface will also be at the height of the eyes of the inspector who checks the integrity of the structure. The vertical column consists of three redundant tubes.

8.2 *Application to the high-mast lighting towers*

The question of wind fatigue is concerning lighting high masts as well as sign bridges. This question is treated for example in [2].

EN40 Standard leaves European States free to regulate the high-mast of lighting towers with the aim to assure a fatigue resistance under the effects of the wind. EN40 Standard does not make any recommendation for fatigue. A CE marked metal mast of candelabrum, is only strictly in conformity with the EN40: it is a certain guarantee of quality, but the CE marked mast does not present with respect to fatigue, i.e. with regard to the dynamic effects of the wind, any particular quality of durability.

The tools which were described above to treat in the specifications of a call for tender the fatigue caused by wind, can also be used for the road lighting candelabra. They will supplement the EN40 standard on this point when it is necessary, whatever the height of the metal mast. You will be able to refer to [1] for more details.

9 CONCLUSION

The storms of 1998 and 1999 bear or does not bear the mark of a climatic change. In any case, they had a beneficial effect by eliminating without fortunately making any victim, the sign bridges which presented already weak points, like fatigue cracks.

Thanks to the publication of Eurocodes, it is now possible for the road managers who wish it, to take account of the risk of cracking of the metal parts -- following stresses of fatigue caused by the wind -- at the first time of the design of new structures.

The simple method presented here can be immediately applied to sign bridges and other structures and equipment of the road, like e.g. anti-noise walls, candelabra, bridge cornices, which all are concerned with the effect of the fatigue caused by the wind. In all the cases the building owner has the responsibility to define the whished service lifespan to be taken into account.

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