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BUCHAREST**

PHD THESIS

**INFLUENCE OF DAMPING SYSTEMS ON THE
CABLES VIBRATION IN CABLE STAYED BRIDGES**

SUMMARY

PhD Student:
Eng. Vlad Daniel URDĂREANU

Scientific coordinator
Prof. Univ. Dr. Eng. Nicolae POPA

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CHAPTER I – CABLE STAYED BRIDGES. OVERALL DESIGN, ADVANTAGES AND DISADVANTAGES

I.1. Introduction

In recent decades, steep increases in freight volumes and the number of people using road infrastructure have been observed both worldwide and in Romania, with forecasts for further growth in the future. To cope with the growing demand, modern transport networks rely on the shortest possible routes, with as few restrictions as possible, to ensure continuous traffic flows and to avoid the formation of traffic jams as much as possible. Basically, there is a lot of emphasis on shortening travel times, which involves new routes which must be as direct and short as possible. This context favors the crossing of obstacles on the most direct and not necessarily the shortest possible variants, thus facilitating the construction of bridges with large and very large spans, among which the most commonly used are suspended bridges and cable stayed ones

In this context, the paper presents a phenomenon specific to cable stayed bridges very often observed worldwide, namely the occurrence of strong vibrations in cables. The paper is a synthesis of some of the most important ones built so far, with a presentation of solutions and trends in the field. Inside the paper, structural deficiencies caused by the uncontrolled vibrations of the cables are highlighted, as well as solutions for their attenuation through various devices, both internal and external. Although the phenomenon has not been noticed so far in Romania, it is more common with the increase of spans and it is possible that in the future new structures susceptible to such vibrations in stays will be erected in our country.

The internal damping of the cables is very small, about 0.5% -1%, compared to the general structure, which is usually between 3-5%, which leads to a large number of oscillations that, if left uncontrolled, can produce damage to both structural and constructive elements around the cables. With the increase of the spans, degradations of the cables and their anchorages were found, caused by vibrations with high amplitudes of the stays, associated with external phenomena of wind and rain.

Există multiple studii pe această temă care este încă dezbătută la nivel internațional. În prezent nu există normative sau ghiduri oficiale pentru inginerii proiectanți în vederea rezolvării acestei probleme. În literatura de specialitate, majoritatea menționărilor ale acestor fenomene au la bază observații vizuale sau au fost deduse în baza (și în urma) degradărilor pe care le-au cauzat. Ca urmare a accesului dificil și a sensibilității echipamentelor, există foarte puține măsurători înregistrate ale acestora la scară reală.

There are many studies on this topic that are still being debated internationally. There are currently no official regulations or guidelines for design that engineers can use to address this issue. In the international literature, most mentions of these phenomena are based on visual observations or have been deduced from (and as a result of) the degradation they had caused. Due to the difficult access and sensitivity of the equipment, there are very few recorded measurements of this on a live scale.

I.2. Evolution

Cable stayed bridges began to be used more and more often after 1950, due to their easier erection, superior aesthetics and also based on economic criteria. The principle of this type of structure consists in crossing large openings with very slender decks by dispersing the efforts in the superstructure in a more favorable way. This is done by taking the loads at multiple points, using cables, and transmitting them to the pylons, which are more rigid and well anchored in the ground. The tension in the cables can be controlled relatively easily, thus being able to induce an optimal internal distribution of stress in the superstructures.

The first cable stayed bridge was built in the 1600s, when the Venetian engineer Verantius designed and erected a bridge supported by diagonal chains, fixed on 2 marginal pillars. The concept was appreciated by engineers, and through experimentation and progress it reached its final version in 1950 in Germany. (Niels J. Gimsing, 1997)

The first modern cable stayed bridge was built in 1956. The Stromsund Bridge in Sweden was designed by engineer Dischinger with a central span of 182.6m and two marginal spans of 74.7m each. The cables were arranged in a fan system, supported by 2 portal-type metal pillars. (Podolny Walter, 1987)

In the years that followed, many other innovative examples emerged in post-war Germany. About 1,500 bridges had been destroyed as a result of the war, which made it necessary for engineers at the time to design structures of larger and cheaper spans to replace what had been lost in the shortest possible time.

Primul pod hobanat realizat în România este cel peste Canalul Dunăre – Marea Neagră, la Agigea, care a fost dat în exploatare în mai 1983. Structura hobanată acoperă trei deschideri de 44.00-40.50-162m, având o lungime totală de 246.50m. Suprastructura podului hobanat este de tip mixt oțel-beton, alcătuită dintr-un tablier

metalic compus din două grinzi casetate, trei lonjeroni și antretoaze, în conlucrare cu o placă monolită din beton. Pentru susținerea suprastructurii sunt prevăzute un total de 10 hobane dispuse în două planuri înclinate în sistem evantai, cu lungimi variabile între 67.00m și 138.00m. Acestea au fost inițial realizate din pachete de cabluri cu câte 48 sârme cu diametrul de 5 mm de calitate SBP I, care au fost ulterior înlocuite cu toroane din câte 7 sârme galvanizate. Podul are un singur pylon având forma literei “A” cu elevație alcătuită din doi stâlpi casetați din beton armat cu secțiuni variabile, uniți la capătul superior. Sub nivelul tablierului, cei doi stâlpi sunt legați cu o riglă casetată din beton armat, care constituie și bancheta de rezemare a tablierului podului hobanat, pe aparate de reazem fix. Înălțimea elevației pylonului este de 78.50 m.

The first important cable stayed bridge built in Romania is the one over the Danube - Black Sea Channel, at Agigea, which was put released to public in May 1983. The cable stayed structure covers three spans of 44.00-40.50-162m, with a total length of 246.50m. The superstructure of the bridge is made of a mixed steel-concrete type, consisting of a metal deck composed of two girders, three longitudinal stiffeners and struts, in conjunction with a poured concrete slab. To support the superstructure, a total of 10 cables are provided, arranged in two inclined planes in a fan system, with variable lengths between 67.00m and 138.00m. These were initially made of cable bundles with 48 wires with a diameter of 5 mm of SBP I quality, which were later replaced with strands of 7 galvanized wires each. The bridge has a single pylon in the shape of the letter "A" with elevation consisting of two reinforced concrete box pillars with variable sections, joined at the upper end. Below the level of the deck, the two pillars are connected with a reinforced concrete box ruler, which is also the support bench for the deck of the bridge, on fixed bearings. The height of the pylon's elevation is 78.50 m. (IPTANA, 2018)

At the time of writing, the largest spans achieved by cable stayed bridges are as follows:

Russky bridge (Rusia)	Sutong bridge (China)	Stonecutters bridge (China)
		
Main span: 1104m	Main span: 1088m	Main span: 1018m
Total length: 1885.53m	Total length: 1690.00m	Total length: 1596.00m
Total width: 29.50m	Total width: 41.00m	Total width: 51.00m
Year of completion: 2012	Year of completion: 2008	Year of completion: 2009

Figure 0-1 – Biggest cable stayed bridges on an international level (Wikipedia, 2020)

CHAPTER II – CABLE VIBRATIONS OF CABLE STAYED BRIDGES

II.1. Phenomena that can induce vibration in cables of stayed bridges

II.1.1. Vortex shedding

In nature, the circulation of air around cylindrical bodies generates a whole series of aero-elastic phenomena that can induce vibrations of considerable amplitudes. The first engineering constructions in which such phenomena were studied are the cooling towers of thermal power plants where, during periods of strong wind, there were recorded displacements at the top with a magnitude of 2-3m. (Chanson H. , 2009) These are due to the phenomenon of vortex shedding, which can also occur in the case of cable bridges.

In fluid dynamics, this is an oscillating flow regime, which occurs when a fluid (such as air) passes through cylindrical bodies at certain speeds. Vortices form behind the object, which periodically detach from either side of it.(Batchelor, 1967)

As the fluid approaches the surface of the cylinder, the pressure level rises from the value corresponding to the free flow to the stagnation point at its tip, corresponding to the highest pressures. The forces generated by the pressure differences tend to push the fluid particles to either side of the cylinder, forming boundary layers. The forces generated on the surface of the body are opposed by the forces given by the viscosity of the fluid, which cannot flow to the back. The boundary layers are detached on both sides, forming 2 shear layers. Particles close to the rigid surface move more slowly than those from the outside, resulting in relative rotations between them, which can lead to vortices. (Benato Roberto, 2017)

Due to their large lengths and insignificant bending stiffness of the cables, they are very sensitive to such phenomena and are prone to resonance. In such situations, the frequency of shedding increases with the average wind speed.

În cazul cablurilor podurilor cu hobane, practica a arătat că pentru anumite structuri există un interval critic al vitezei vântului la care oscilațiile au amplitudini mai mari, iar fenomenul de desprindere a vârtejurilor persistă mai mult timp și are un efect mai pronunțat. Acest interval este intitulat zona de fixare și este de ordinul a 10-20 m/s.

In the case of cable stayed bridge cables, practice has shown that for most structures there is a critical range of wind speed at which the oscillations have higher amplitudes, and the phenomenon of vortex shedding persists longer and has a more pronounced effect. This interval is called the fixation area and it has a magnitude of 10-20 m/s. (Hikami, 1988)

II.1.2. Burst vibrations

Toate aspectele prezentate în capitolul anterior au la bază ipoteza că viteza vântului rămâne constantă în timp, dar în realitate aceasta poate varia chiar foarte mult în intervale relativ mici de ordinul secundelor. Cum forța ce acționează asupra cablului este direct proporțională cu viteza vântului, aceste variații în timp generează răspunsuri dinamice în hobane.

All the aspects presented in the previous chapter are based on the hypothesis that the wind speed remains constant over time, but in reality, this can change abruptly in relatively small intervals of magnitudes of seconds. As the force acting on the cable is directly proportional to the wind speed, these variations over time generate dynamic responses in the cables. (Sena Kumarasena N. P., 2007)

II.1.3. Wind-rain combination (Sena Kumarasena N. P., 2007)

Combinăția dintre ploaie și vânt cu viteză moderată poate cauza oscilații în cabluri cu amplitudini și perioade de vibrație mari. Acest fenomen a fost observat la multe poduri hobanate și a fost studiat intens.

De-a lungul timpului au fost realizate o sumedenie de cercetări asupra acestui fenomen, incluzând și măsurători în teren, teste în tunelul de vânt sau modele analitice. S-a constatat că aceste oscilații apar în special atunci când există ploaie și vânt cu viteză moderată între 8-15 m/s, pe o direcție de 20°-60° față de planul cablului, cu hobana înclinată spre direcția vântului.

Perioadele înregistrate ale oscilațiilor cablurilor au fost mai mari de 0.33s, iar amplitudinile maxime între 0.25m-1.00m, rezultând mișcări violente care au dus chiar la lovirea între ele a hobanelor adiacente.

Fenomenul de inducere a vibrațiilor în hobane în urma combinației vânt-ploaie reprezintă unul din cele mai importante mecanisme de care trebuie să se țină seama în proiectarea podurilor cu hobane și a sistemelor de amortizare a vibrațiilor, datorită amplitudinilor mari ale oscilațiilor rezultate și aparițiilor foarte dese.

The combination of moderate speed wind and rain can cause oscillations in cables with high amplitudes and vibration periods. This phenomenon has been observed in many cable stayed bridges and has been studied extensively.

Over time, a lot of research has been done on this phenomenon, including field measurements, wind tunnel tests or analytical models. It has been found that these oscillations occur especially when there is rain and wind with a moderate speed between 8-15 m / s, in a direction of 20°-60° from the plane of the cable, with the cable inclined towards the wind.

The recorded periods of cable oscillations were greater than 0.33s, and the maximum amplitudes between 0.25m-1.00m, resulting in violent movements that even led to the collision of adjacent stays.

The phenomenon of inducing vibrations in cables via the wind-rain combination is one of the most and important mechanisms to be taken into account in the design of cable stayed bridges and vibration damping systems, due to the large amplitudes of the resulting oscillations and very frequent occurrences.

II.1.4. Wake galloping

Wake galloping is a phenomenon that induces oscillations in stays which occurs due to the influence of other objects around the cable. These objects can be adjacent constructions or bridge pylons, but most often they are adjacent cables. The oscillations of one cylinder downstream can be induced by the turbulent flow regime in the wake of another located upstream.

II.1.5. Ice deposits

Ice deposits or snow on the cables is similar to the phenomenon of inducing oscillations in the wind-rain combination, but at much lower temperatures. Due to wind, precipitation and low temperature, deposits (snow or ice) begin to accumulate on the outer surface of the cable.

II.1.6. Traffic actions

During service, bridge superstructures are constantly subjected to vibrations caused by traffic, especially railroad. These are due to the dynamic effect of the vehicles and the interaction with the road. In the case of cable stayed structures, vibrations can be easily transferred to the cables by means of anchors. Given the wide range of loading positions and variations that can occur, it is plausible to assume that the oscillations of the cables caused by traffic can resonate. This effect occurs especially when one or more of the superstructure's own vibration periods are close to those of the cables.

II.2. Parameters used for studying cable vibrations in cable stayed bridges

II.2.1. Reynolds number

It is a dimensionless quantity used in fluid dynamics to predict the characteristics of a flow and especially of the regime (laminar, turbulent or transient) around an object. The concept was first used by Gabriel Stokes, but it was Osborne Reynolds who popularized the concept. It is defined as the ratio of inertial to viscous forces, quantifying the relative importance of these 2 sources in determining the type of flow. (Acheson, 1990)

$$R_e = \frac{\rho \cdot v \cdot D_c}{\mu} \quad (\text{II.1})$$

Where D_c – characteristic length

ρ – density of fluid

v – flow speed

μ – fluid dynamic viscosity

For small values of this number, the flow is laminar (smooth, constant), and viscous forces predominate. At high values, inertial forces that tend to produce chaotic flows (eddies, vortices, etc.) begin to dominate, leading to turbulence and instability of the flow regime. (Acheson, 1990)

II.2.2. Strouhal number

It is a dimensionless quantity that describes the oscillations of the periodic flow mechanisms. It is named after the Czech physicist Vincent Strouhal, who conducted many experiments to analyze phenomena such as vortex shedding the so-called "singing in the wind". (Hikami, 1988)

$$S_t = \frac{f \cdot L}{v} \quad (\text{II.2})$$

Where f – frequency of shedding, L – characteristic length, and v – flow speed

For cylinders, the Strouhal number can be considered 0.2 for a wide range of flow rates. In this situation it is observed how the increase of the speed leads to the decrease of the period of shedding of the vortices. Thus, the alternation of vortices induces an almost harmonic transverse force and can induce resonance phenomena as the period of oscillations approaches one of its own periods of vibration of the structure corresponding to the displacement in that direction. Changing the speed changes the shedding frequency and can even resonate on other frequencies as it approaches any of the dominant frequencies of the structure. (Hikami, 1988)

II.2.3. Lift coefficient and drag coefficient (Batchelor, 1967)

In fluid mechanics, the drag coefficient (usually denoted C_D) is a dimensionless unit used to determine the drag of an object in a fluid environment. It is used to determine the force exerted by the moving fluid around the object in the direction of flow. (Batchelor, 1967)

$$C_D = \frac{F_D}{\frac{1}{2} \rho \cdot v^2 \cdot A} \quad (\text{II.3})$$

Where F_D – force induced by the fluid on the object, along the direction of the flow, ρ – fluid density, v – flow speed, and A – reference area

The lift coefficient (generally denoted by C_L) is also a dimensionless unit which, unlike the drag coefficient, is used to determine the lift force exerted by the fluid while passing around the object (perpendicular to the direction of flow).

$$C_L = \frac{F_L}{\frac{1}{2} \rho \cdot v^2 \cdot A} \quad (\text{II.4})$$

Where F_L – force induced by the fluid on the object, perpendicular to the direction of the flow

II.2.4. Scruton number

It is a dimensionless parameter very often used in fluid mechanics to determine the damping of the structural response induced by the wind. In general, the damping of wind-induced instability is considered based on this number. (Hikami, 1988)

$$S_C = \frac{m \cdot \xi}{\rho \cdot D^2} \quad (\text{II.5})$$

Where m – mass pe unit, ξ – fraction of the critical damping of the structure, D – obstacle width, and ρ – fluid density

In general, theoretical studies and cable observations have shown that for values of the Scruton number above 10, the vibrations induced by the usual phenomena to which they are exposed are damped and do not reach critical values that endanger strength or structural stability. (Hikami, 1988)

Usually, out of the 4 parameters that make up the Scruton number, only damping can be modified by engineering solutions, the others being fixed for other reasons. The mass of the cable depends on the number of strands resulting from the ultimate limit state calculus, the outer diameter results depending on the number of strands, so that they fit inside the protective sheath, and the air density is implied by the area where the brodge is located, but does not usually vary.

The engineering practice for vibrations damping of the stays involves determining the Scruton number and, depending on its value, the introduction of energy dissipation devices to increase the damping. As a general rule, short cables do not require any device, but as their length increases, it becomes necessary to adopt various solutions to increase the damping.

II.3. General solutions and systems used for damping of cable vibrations

II.3.1. Cable sheath profiling

In order to reduce the vibrations caused by the wind-rain combination, various embossed profiles are made on the outside of the cable sheaths, which disturb the drain and prevent the formation of water rivulets. They do not completely stop the phenomenon, but substantially reduce its effects.

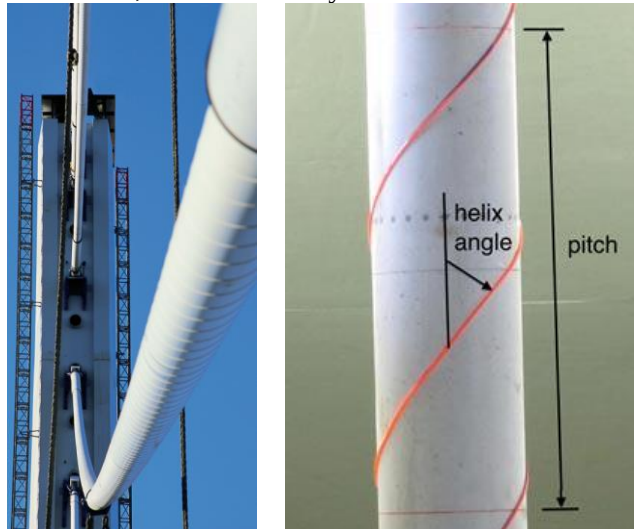


Figure II-1 – Profiled sheaths (Wind tunnel investigations of an inclined stay cable with a helical fillet, 2014)

The most commonly used profile is in the form of a helical wire on the outside of the sheath which continues along the entire length of the stay. It is both easy to make and maintain over time. Studies are currently underway to find more efficient profiles.

II.3.2. Cross ties

They are transverse cables that create connections between the main stays or between them and the superstructure at key points. The main purpose is to change the deformed shape of the cable to dampen the vibrations over a certain mode of vibration. As a favorable side effect, the fact that the attachment points are on adjacent hobs that each have different dynamic characteristics causes them to interact with each other and stop each other from resonating.

The first and best-known example of a cable stayed bridge at which this system was used is the Normandy Bridge over the River Seine in France. It has a main span of 856m, a total length of 2143m and a width of 23.60m. From each pylon to the center of the spans there are 3 sets of cross ties arranged at equal distances. Their purpose is to bring the oscillations of the cables to the 4th mode of vibration.



Figure II-2 – Cross ties on the Normandy bridge (Wikipedia, 2019)

Cross ties create rigid links between stays. Their role is not to effectively dampen vibrations, but to change their own periods and modes of vibration, favoring the higher ones that have lower amplitudes. In applying this solution, it must also be taken into account that too many transverse cross ties can become inaesthetic.

II.3.3. Tuned mass dampers

II.3.3.1. Tuned mass dampers with elastic links

The vibration damping system based on tuned masses involves fixing weights at certain points of the cables. They aim to change the dynamic characteristics of the cable (its own modes, periods and vibration frequencies) in order to avoid the resonance phenomenon.

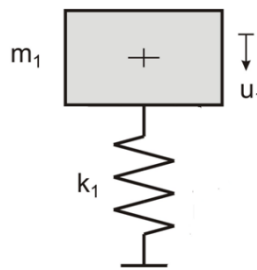


Figure II-3 – Mathematical model for tuned mass dampers (Habib Tabatabai, 2003)

The main advantages consist of:

- high durability over time, as all components are mechanical and robust
- insensitivity to strong temperature variations
- relatively low costs for both the device itself and its maintenance

The main disadvantages:

- the fact that they are effective only for certain intervals of amplitudes and periods of oscillations, for which they have been calculated
- difficult to handle on site because they are relatively heavy and need to be placed in hard to reach locations (usually in the middle of cables)

The first models of such devices were made of 2 concrete blocks connected by a rod which in turn was fixed to the cable at a distance that allowed relative movement between it and the weights.

Nowadays, steel masses are used instead of concrete blocks. They are hollow inside and can have different geometric shapes such as: sphere, cylinder, cone, bell, etc. Newer models have uneven weights, which allows them to operate efficiently over a wider range of frequencies.

The positioning of the masses along the cable also depends on its own vibration mode. Zones of maximum amplitude or inflection are usually avoided. The chosen location should theoretically increase the number of half-waves that result from vibration and thus lead to a decrease in maximum displacements.

When calculated properly, built and installed properly, the masses efficiently absorb unwanted energy from the cable and even dissipate it internally, thus significantly reducing deformations and with it, early degradation that may occur.

II.3.3.2. Tuned mass dampers with viscous links

These represent a relatively new technology that involves the introduction of a hysteretic viscous energy dissipation mechanism inside the tuned mass. They have not yet been implemented in any structure, but there is a lot of promising research on the subject.

Although the dissipation mechanism loses its efficiency due to the lack of connection with a rigid element in which the resulting forces can be discharged, the much better position along the cable gives an increased efficiency to this type of device. At the moment this type of solution is still in its infant stages.

II.3.4. External dampers

II.3.4.1. The principle of external dampers

The most commonly used method to ensure a sufficiently high level of damping in cable stays, so as to avoid degradation of structural elements, is the attachment of external transverse dampers. Their role is to dissipate energy from the cables and reduce vibration amplitudes to an acceptable level. Ideally, they should be fixed as close as possible to the middle of the cables, but due to the physical limitations, they are attached close to the anchorages, at about 1% -3% of the cable length.

These devices, also called energy dissipators, work by generating a force in the damper in response to deformations due to displacements in the cable. The product of the two generates mechanical work, which thus dissipates energy and reduces the amplitudes of vibrations. The energy dissipation capacity per cycle is expressed in the damping hysteresis. The larger the area bounded by it, the larger is the energy dissipation capacity.

Depending on the function mechanisms, they can be divided into 3 main categories, in order of their efficiency, as follows:

- elastomeric dampers - are based on the ability of the special rubber in their composition to deform in two modes, an elastic one followed by a plastic one
- viscous dampers - are based on the force generated when a special fluid passes through a piston with special holes at the end
- friction-based dampers - are based on the force generated on a friction surface when it moves

II.3.4.2. Elastic and friction dampers

Elastic dampers reduce cable oscillations by storing and dissipating energy internally in the component materials. This is done by mechanical work given by elastic internal forces but especially plastic ones. They work by the deformation under shear force of a special elastomer, with a well-defined elasto-plastic behavior.

A similar function mechanism can be considered in the case of friction-based dampers which, by means of internal pre-compression forces, generate response forces following the movement of the cable. The pre-compression force is absolutely necessary and quite difficult to maintain. Practice has shown that the very high number of repetitions degrades this type of device prematurely, which is why they are not used very much in the case of cable stays.

Their main advantage over other dampers is their smaller size and easier installation. Their disadvantage is that they can only accept small deformations, which greatly diminishes their ability to dissipate energy.

The fact that all their damping characteristics are dependent on the properties of the component materials makes these dampers very dependent on the outside temperature. The elastomer becomes very flexible at high temperatures and very rigid at low temperatures, a sudden decrease can even lead to its rupture. Also, in the case of those based on friction, the coefficient of friction between the 2 materials is strongly influenced by the outside temperature and the humidity of the environment.

Calculation formula for the force generated by an elastic damper:

$$F = y_d \cdot K_{eff} = y_{el} \cdot K_{el} + (y_d - y_{el}) \cdot K_{pl} \quad (II.6)$$

Where: y_d, y_{el} – total displacement, respectively elastic displacement

K_{eff}, K_{el}, K_{pl} – effective stiffness, elastic stiffness, plastic stiffness

II.3.4.3. Viscous dampers

Viscous dampers are devices consisting of cylinders with pistons that have holes in the head through which a special fluid is able to pass, which dissipates energy.

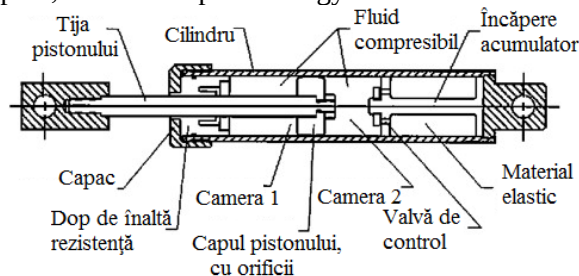


Figure II-4 – Sketch of a viscous damper (Main, 2002)

Such devices consist of 2 chambers, separated by the piston head, in which there is a compressible liquid based on silicone. The holes allow fluid to flow from one chamber to another as the piston moves. The force is

generated as a result of the pressure difference between the piston head and the compression in the fluid. (Main, 2002)

As such, the force generated by the damper is dependent on the speed of the piston and not on its deformation.

The mathematical model underlying the computational relations consists of a spring of rigidity K , inserted with a piston of viscosity C . In addition, there is another important parameter, which determines the influence of velocity on the freedom of movement of the liquid from one room to the other, which is further denoted by α . Depending on this coefficient, the curvature in the displacement force diagram will be smaller or larger. According to laboratory records, the extreme plausible values for this parameter are 0.15 and 2.

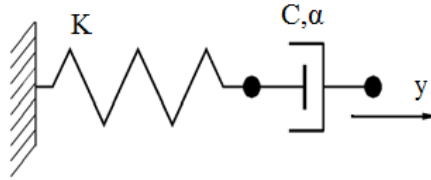


Figure II-5 – Viscous damper mathematic model (Main, 2002)

The calculation formula for the force generated by a viscous damper is:

$$F = K \cdot y_d + C \cdot v_d^\alpha, \quad (\text{II.7})$$

Where K is the linear stiffness of the damper, y_d displacement of the piston head, C viscous coefficient of the fluid, v_d speed at the head of the cylinder, α speed exponent, characteristic to the liquid and the shape of the holes inside the cylinder.

Based on past performed studies, the international literature specifies the calculation formula for the optimal viscosity coefficient depending on the specific vibration mode targeted (Main, 2002):

$$c_i^{opt} = \frac{1}{i \cdot \pi \cdot \frac{x_d}{L}} \cdot \sqrt{T \cdot m}, \quad (\text{II.8})$$

where i is the number of the acquired vibration mode

x_d distance along the cable measured from the end to the position of the damper

L total length of the cable

T tension in the cable

m linear mass of the cable

c_i^{opt} – optimal viscous coefficient corresponding to vibration mode “ i ” of the cable

II.3.5. Semiactive dampers - magneto-rheological

Magneto-rheological dampers are semi-active dissipators that can change their properties using magneto-rheological fluids. These fluids have the ability to change their state from liquid to semi-solid in a continuous and completely reversible manner by means of a magnetic field. (Erik A. Johnson, 2007)

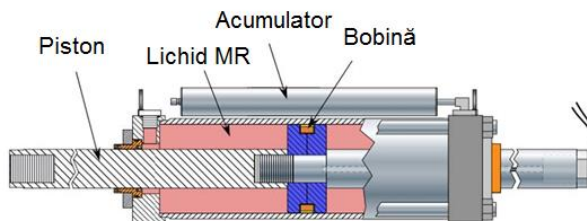


Figure II-6 – Schiță amortizor magneto-reologic (Maurer, 2020)

Magneto-rheological fluid is a special oil, which, when introduced inside a magnetic field, changes its apparent viscosity until it reaches a certain visco-elastic stage. More importantly, its resistance can be controlled with great precision by varying the intensity of the magnetic field.

The mathematical model underlying the force calculation in the damper is more complex than the rest, involving several components. It should be noted that all the constants from the other models, such as viscosity or elastic stiffness, are here variables that depend on the intensity of the electric current that generates the magnetic field.

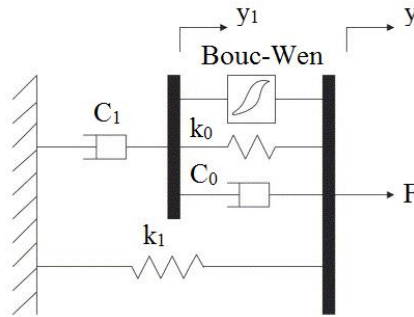


Figure II-7 – Mathematical model of a magneto-rheological damper (Erik A. Johnson, 2007)

II.4. Examples of problems and structural degradation caused by cable vibrations

II.4.1. Dubrovnik bridge

The Dubrovnik Bridge crosses the Dubrovacka River to the western entrance to the town of the same name and is the only cable stayed bridge in Croatia. It is an asymmetrical bridge, consisting of a frame structure made of console with a side span towards Split of 87.35m and a console to the central span of 60.05m, made up of a concrete box section, continued with an asymmetric cable stayed structure, with a single pylon and a length of 244.00m in the central span and a side one of 80.70m towards Dubrovnik, made up of a mixed steel-concrete section. The 2 structures are connected by an articulated link. (Z. SAVOR, 2006).

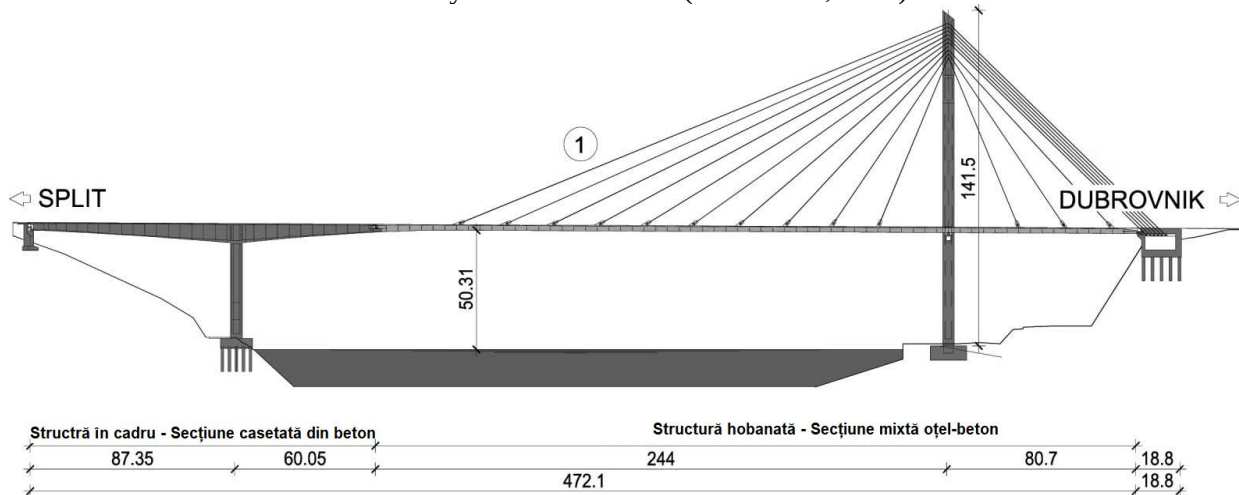


Figure II-8 – Longitudinal section of the Dubrovnik bridge (Z. SAVOR, 2006)

Longitudinally, the cables are arranged in a modified "fan" type system. In the transverse direction, the cables are arranged in 2 inclined planes and consist of bundles of galvanized parallel strands, protected in 225mm diameter HDPE sheaths injected with wax. The number of strands in each stay varies between 27-61, and the longest 6 cables are 134.80m, 152.40m, 170.50m, 188.90m, 207.70m, respectively 226.70m (Z. SAVOR, 2006).

Shortly after the opening of traffic in 2002, during a light rain combined with moderate wind speed between 12.9-14.7m/s, strong vibrations of the longest cables in the main span were observed. The longest 3 cables were excited, and the vibrations stopped only 20 minutes after the rain stopped. There were 12 similar incidents the following year with high-amplitude and low-frequency cable vibrations during light rains accompanied by medium-speed winds. Following each incident, visual inspections of the stays and their anchorages were carried out and, as no deterioration was observed, no precautionary measures were deemed necessary. (Z. SAVOR, 2006).

The in March 2005, during a heavy snowstorm, almost all the stays began to vibrate violently, especially in the main span. These vibrations were accompanied by a loud thud caused by the friction of the strands between them. Movements of the superstructure characteristic of the phenomena of bending combined with torsion were also observed. These oscillations lasted about 2 hours, and from the analysis of the video recordings it was concluded that the longest cables oscillated mainly in the vertical plane with a frequency of 1.27Hz, corresponding to their second own mode of vibration. (Z. SAVOR, 2006).

The oscillations were so strong that 2 cables collided with the lighting poles, knocking them down and locally degrading the HDPE protection sheaths (Figure II-9). The minimum distance between cables and poles was 85cm, so maximum amplitudes of approximately 2.50m could be estimated. One of the protective sheaths broke near a joint, leaving the cable exposed for a length of about 7.50 m (Figure II 9). Also, a significant number

of the M24 prestressed bolts near the anchorages broke. Similarly, the anchors at the level of the superstructure were similarly degraded, but in this case the bolts only loosened and did not break. (Z. SAVOR, 2006).

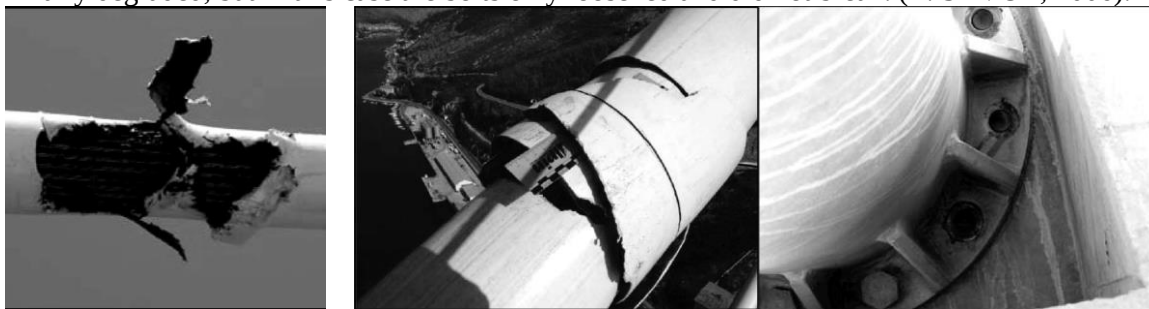


Figure II-9 – Damage to the protective sheath and destroyed bolts in the anchorage area (Z. SAVOR, 2006)

A year later, in May 2006, another heavy snowstorm struck the bridge, causing strong vibrations to the longest 6 cables of the bridge in the main span, accompanied by movements of torsional bending of the superstructure, similar to the first incident. The oscillations lasted about 6 hours. The previously repaired protective sheath broke along the weld falling into the sea, and much of the replaced screws were broken again, having been found on the superstructure. (Z. SAVOR, 2006).

For the rehabilitation procedure, the vibration damping solution by means of external dampers attached to the cables was chosen. Magneto-rheological dampers were used, mounted on the 6 longest cables on the main span and on all the cables on the secondary one. Their role is to reduce vibrations only in the vertical plane, considering that no additional damping is needed in the horizontal plane. Although their installation began after the first major incident in 2005, the works lasted more than a year, being carried out under traffic, thus allowing for the second incident. However, no such events were recorded after the works were completed (Z. SAVOR, 2006).

II.4.2. „Veterans Memorial” și “Fred Hartman” bridges – Texas, U.S.A. (Felix Weber G. F., 2006)

The two cable stayed bridges in the state of Texas - U.S.A. were completed in 1991 and 1995, respectively, and were the subject of an elaborate investigation into cable vibration, which culminated in 2004 with the installation of viscous dampers on both structures.

The "Veterans Memorial" bridge is located in the port of Arthur and has a total length of 380m, with 3 spans of 92.50-195.00-92.50m. The cross section of the deck consists of a concrete box with a width of 17.00m. The pylons are of the “H” type, with a total height of 95.00m and a total of 112 cables placed in 2 vertical planes and arranged in a harp system.

The “Fred Hartman” Bridge crosses the Houston Canal in La Porte and has a total length of 4km, of which the cable stayed structure is 754m, being made up of 3 spans of 187.00-380.00-187.00m. The cross section of the deck is of mixed steel-concrete type and consists of girders linked by crossbeams and a concrete slab. The pylons are of the “diamond” type with 192 cables placed in 2 vertical planes and arranged in a fan system.

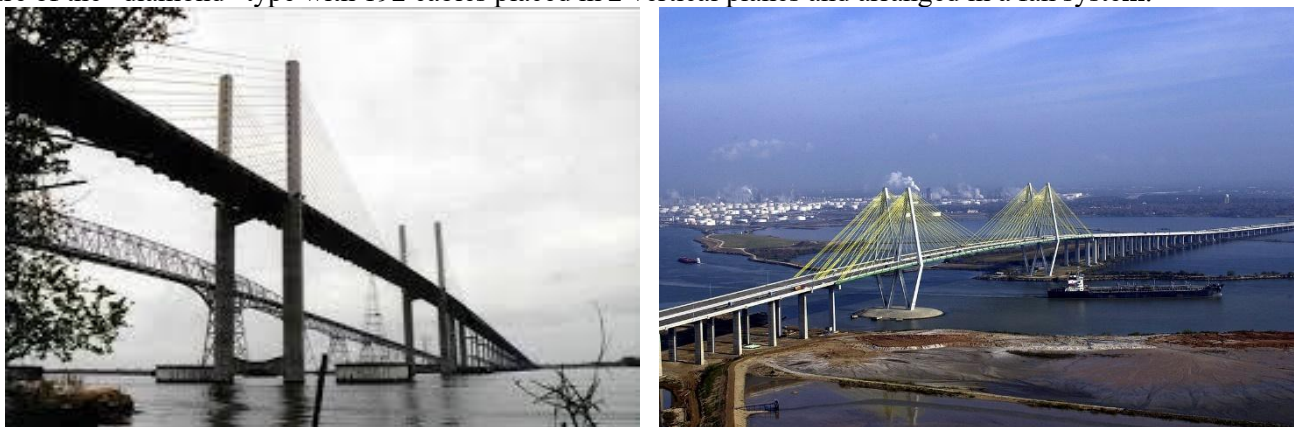


Figure II-10 – „Veterans Memorial” and “Fred Hartman” bridges (Wikipedia, 2021)

Since 1997, a process of careful monitoring of the 2 structures has begun, with strong vibrations of the cables being observed during rains. These oscillations reached peak amplitudes of over 10 times the diameter of the stays in some of the cables, the movement usually taking place after one of the 3 main modes of vibration.

From the general meteorological conditions that induced these strong vibrations, 2 main characteristics were noticed, determined as defining, namely the weak to moderate intensity of rain and wind. Oscillations were not recorded during heavy rains or winds.

Up to a certain level, the vibrations induced by the wind-rain combination at the 2 structures did not seem to be influenced by the length of the spans or the cables. Although one of the bridges has a span larger by twice than the other, both bridges showed strong oscillations of the stays and significant degradation of their protection systems. However, the amplitudes recorded on the "Fred Hartman" bridge had a higher degree of severity, one of the explanations being the greater flexibility of the superstructure which seemed to transfer a large part of its own energy to the cables.

Periodic inspections revealed more than 100 broken welds of the cable's sheaths, but also to certain details of the anchorages, probably caused by large deformations and fatigue. More detailed measurements also revealed major problems with the corrosion protection layer injected into the stays, reducing the durability and corrosion resistance of the cables to a dangerously low level.

According to (Caetano), multiple scenarios for securing structures against vibrations have been proposed, of which 2 were initially chosen, each having been tested locally on bridges, in order to establish the most effective one.

The first scenario proposed connecting the cables with a set of cross ties, in order to reduce their own vibration periods below 0.5s and to increase the internal damping.

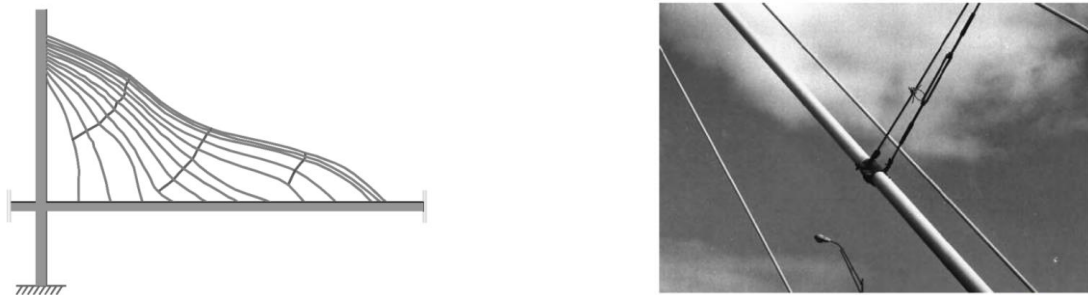


Figure II-11 – Cross ties used for stiffening the stays (Caetano)

Although this system was tested on structures and showed that it works as expected, the solution finally adopted for both bridges, based on a much higher efficiency, was to install viscous dampers at the superstructure level on all cables.

According to (Caetano), this system has led to a significant increase in damping, which has made it possible to reduce the amplitudes of the cable oscillations much more efficiently and over a longer period of time. The final assembly was made more than two years after the first device was installed and tested. Following the completion of the works, no problems related to the vibrations of the cables were recorded at any of the two bridges, although their monitoring continues to this day.

II.5. Examples of using vibration damping systems

II.5.1. Normandy bridge

The Normandy Bridge, inaugurated in 1995 in France and links the regions of La Havre and Honfleur. It is the first cable stayed structure for which the effects of cable vibrations have been considered since the design phase. With a main span of 856.00m and a total length of 2141.00m, it set the record for the largest span for cable stayed bridges at the time of completion.

The "A" shaped pylons have a total height of 214.77m, and the deck is approximately 60.00m above the water level, resulting in a maximum cable length of over 450.00m. To reduce their vibrations, cross ties were used, arranged in 4 planes on each span along the longest stays, tying all the cables to the level of the deck. At the time, these were state-of-the-art technologies in the field and had not been applied anywhere else.



Figure II-12 – Normandy bridge and its cross ties (Jacobs, 2012)

Soluția s-a dovedit a fi una de succes, până în prezent structura neavând nici un fel de probleme din acest punct de vedere. La momentul dării în exploatare, podul a atins 2 recorduri, dintre care unul pentru cea mai mare deschidere și al doilea pentru prima structura cu sisteme de atenuare a vibrațiilor cablurilor prevăzute încă din faza de proiectare.

The solution proved to be a successful one. So far, the structure has not had any problems regarding cable vibrations. At the time of commissioning, the bridge achieved two records, one for the largest span and the second for the first structure with cable vibration damping systems provided since the design phase.

II.5.2. Sutong bridge

With a main span of 1088m and two side spans of 300m each, Sutong Bridge set the record for the largest span of a cable stayed bridge in 2008. It crosses the Yangtze River and connects the cities of Suzhou and Nantong in China, with a total length of 8206m.

The "A" shaped pylons have an impressive height of 306m, and the cables reach up to 541m in length. In order to ensure all the criteria of strength and stability, a series of tests were performed on the behavior of the stays, some of which were even done on full scale. Following this, a complex cable vibration damping system consisting of passive and semi-active dampers was installed.

The Sutong Bridge has a number of 272 stays with lengths between 154m and 541m. It was decided to attenuate the vibrations with passive viscous dampers at cables numbers 10 ($L = 228\text{m}$, $T = 1.785\text{s}$) up to 28 ($L = 472\text{m}$, $T = 3.571\text{s}$), and with magneto-rheological dampers for the longest 6 stays, respectively numbers 29 ($L = 483\text{m}$, $T = 3.584\text{s}$) to 34 ($L = 541\text{m}$, $T = 4.002\text{s}$).



Figure II-13 – Dampers installed on the Sutong Bridge, China (Felix Weber H. D., 2014)

The devices were installed perpendicular to the cables in their vertical plane, at about 3.50m above the deck level. The anchorages are located at 1.30 m below the level of the track, thus resulting a relative position of the devices of approximately 2.30% along the cable's length. Each stay has an individual sensor installed that transmits data to a central control unit, positioned inside the deck that can control all 12 dampers. In addition, each device is connected to external batteries, capable of operating for at least 48 hours after power failure.

Similar systems have been used in the cases of the Franjo Tudjman Bridges near Dubrovnik, Croatia, or the Alamillo Bridge in Seville, Spain, where it has been implemented as a retrofitting measure.

The test of the system was performed on a physical model in 2007, on a cable similar to the one at Sutong Bridge, with a length of 228m.

The test results showed that both the calculation assumptions and the control systems worked correctly, recording a level of damping of 7.5%. The influence of the actual stiffness of the cable, which proved to be lower than the theoretical values, and the rotation of the anchors, which turned out to be higher than had been considered in the calculation hypotheses, were also highlighted.

II.5.3. Russky bridge

The Russky Bridge near Vladivostok, which connects Russky Island and the Muravyov-Amursky Peninsula, completed in 2012, currently holds the world record for the largest span of a cable stayed bridge with a length of 1104m.



Figure II-14 – Russky bridge (<https://structurae.net/en/structures/russky-bridge>)

The structure has a total length of 1885.50m, made up of multiple spans of 60-72-3x84m-1104-3x84-72-60m.

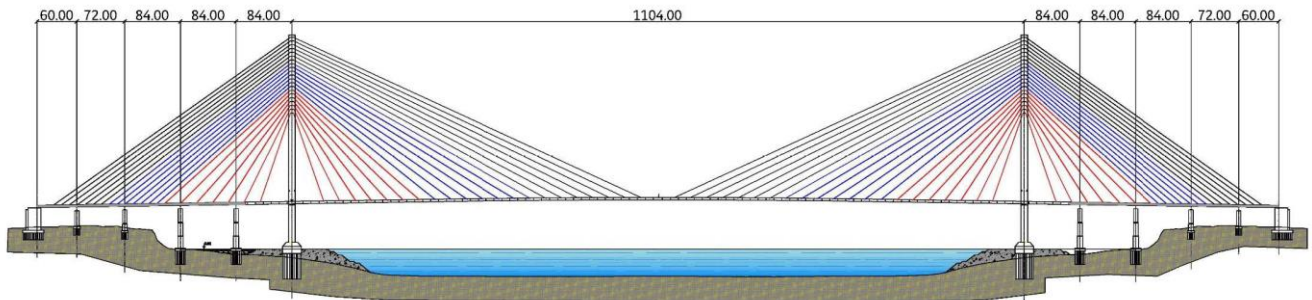


Figure II-15 – Sketch of the Russky bridge (Mellier, 2014)

The “A” shaped pylons have a total height of 319m and have 3 stiffening beams, one of which is at the level of the deck. The cable stayed structure contains a number of 168 cables arranged in a fan system with lengths between 135m and 582m, also holding the record for the longest stay.

In order to ensure stability and maintain the bridge in operating conditions even during the most severe storms, for which average wind speeds of up to 36m/s have been taken into account, a complex vibration damping system has been installed and monitored with 48 accelerometers and 48 pressure cells per pylons, which consists of:

- passive viscous dampers having been fitted on the shorter cables with numbers from 1 to 16
- at the longest hobs from 17 ($L = 483\text{m}$, $T = 3.571\text{s}$, $a/L = 2.6\%$) to 21 ($L = 580\text{m}$, $T = 4.165\text{s}$, $a/L = 2.3\%$) magneto-rheological dampers were installed (where a/L is the relative position of the device along the cable)

Unlike the Sutong Bridge, in order to reduce power consumption, the control unit was installed for each device separately and accelerometer sensors placed directly on the cables at a height of about 28.00m from the deck level. Also, the devices used at this bridge include two dampers in each position, in order to be able to also reduce the horizontal vibrations. However, the biggest innovation in this system is that the magneto-rheological dampers also have temperature sensors and can make the necessary corrections to ensure the optimal parameters over a wide temperature range between -40°C and $+60^\circ\text{C}$.

CHAPTER III – VIBRATION ANALYSIS BY NUMERICAL METHODS

III.1. General

As in many other cases in the field of constructions, the study of cable vibrations involves design situations with many parameters that vary over time and with mathematical problems of very high level of complexity that cannot be solved by analytical methods. The best engineering approach to solving these problems is obviously one based on numerical methods, and more precisely, finite element calculation methods.

However, this approach has a series of obstacles that must be overcome, starting from the formulation of the elements used, discretization and even the type of analysis done. The phenomena that must be modeled mathematically belongs to two distinct branches of civil engineering, namely fluid mechanics, which is necessary to simulate the external environment in which the cable moves, respectively the structural mechanics that must determine the internal state of stress and deformations in the cable.

The discretization must be as fine as possible to be able to reproduce second-order effects in the cable, but the modeled space must be large enough to cover the aeroelastic phenomena that develop around it, resulting in a very high number of finite elements.

In order to be able to have a sufficient level of accuracy for the variation of effort-deformation at the cable level and to be able to draw conclusions about the type of oscillations that occur in it, the type of analysis adopted cannot be static, and so, must be dynamic. The modeled cables are expected to exhibit geometric nonlinearities, and the air circulation to describe movements with a high degree of turbulence.

III.2. Numerical models used to simulate cable vibrations

III.2.1. Problems encountered in modelling cable vibrations

The main problems encountered in modeling the dynamic response of a cable stay is the complexity of the analysis itself that requires high computational power and the uncertainty regarding the level of the accuracy of the obtained results.

The most accurate simulation of cable vibrations under the action of external forces would involve a discretization of the cable with finite elements specific to structural mechanics, a discretization of the space around the cable with finite elements specific to fluid mechanics and simulating the interaction between the 2 types of elements, all of this inside a time-history analysis with a high degree of nonlinearity. This method involves a very laborious calculation that will not always generate the best results. With such a high degree of complexity, the results need to be validated on the basis of scale models, which often cannot accurately reflect the dynamic response due to large discrepancies between the actual dimensions of the cables reaching hundreds of meters and the scaled models, which cannot usually be longer than a few meters due to practical limitations.

III.2.2. Proposal of a simplified method for simulating cable vibrations

III.2.2.1. Presentation of the calculus algorithm

Following those presented in the previous chapters, it can be stated that the simulation of cable vibrations involves a high degree of complexity that can be the subject of many research papers. One of the objectives of this paper is to find a simplified method of performing this analysis, more accessible to the engineers involved in the conception or design of structures with cable stays. The idea is to introduce a series of simplifying hypotheses to make these analyzes more accessible and to compare them with more detailed analyzes, as well as scale models, in order to confirm the hypotheses and the accuracy of the results obtained.

A first simplifying hypothesis is to limit the interaction between the air and the sheath by neglecting the local effects on it, which is usually limited to the sheath and not on the cable. In this case, it is proposed to model the excitation that induces vibrations in the cable stay by means of an external force with sinusoidal variation that acts directly on the resistance cables.

Another hypothesis is to abandon the modeling of cable stay with complex finite elements that contain mathematical formulations for the complex behavior of the cable, in favor of a model with multiple finite elements of bar type, with nodes at both ends, each with 6 degrees of freedom, similar to the chain theory in the literature.

The vibration damping devices will be modeled by means of special finite elements with hysteretic behavior such as viscous dampers.

Regarding the internal damping, it is proposed to model by the Rayleigh method for a damping of 0.5% on the first 5 modes of vibration. This value was chosen based on similar studies in the field such as (Erik A. Johnson, 2007) (Felix Weber H. D., 2014) (Y. L. Xu, 2003) etc., as being sufficient and corresponding to the most unfavorable scenarios.

Regarding the type of analysis, the possibility of adopting time-history type analyzes with modal overlap in favor of those with direct integration will be studied. By doing this, it must be assumed that in modal analyses the effects of geometric nonlinearity will be lost.

In order to establish the accuracy of this approach and to rectify the considered parameters, a small-scale model was made in a wind tunnel. The study is presented below.

III.2.3. Validation of the algorithm through experimental studies in the wind tunnel

III.2.3.1. Presentation of the scale model and measurements performed

The aim of this study is to determine the dynamic response of a tilted cable under the effect of the wind acting perpendicular to the plane it forms with its projection on the ground and the possibility to extrapolate these results to simulate as accurately as possible the behavior of real cable stays by the finite element method.

The tests were performed in the wind tunnel of the Faculty of Hydrotechnics within the Technical Constructions University of Bucharest. The tunnel has a length of 25.00m and a square section with a side of 1.75m. The walls of the tunnel are mostly made of 2mm sheet metal and 10mm thick plexiglass panels for visibility in the relevant areas, fixed on the edges with screws of a metal structure. The air inside is driven by a propeller positioned at the downstream end of the tunnel. At the opposite end, where the air intake is located, there is a steel grid with square mesh of about 1 cm with the role of laminating the wind flow.

The cable used for the tests has a section composed of 3 wires with a diameter of 3mm (3 $\varnothing$ 3) and the conventional material yield limit $f_t = 1860 \text{ N/mm}^2$. It is the same cable used for pre-stressing railway supports. The cable was fixed to the walls of the tunnel diagonally, in the opposite corners to be as long as possible. In the upper corner it was attached by means of a fixed (passive) anchorage, and in the lower one by a mobile (active) anchorage made up of a mechanical jack with a capacity of 4 tons linked to the ground.



Figure III-1 – Passive anchorage

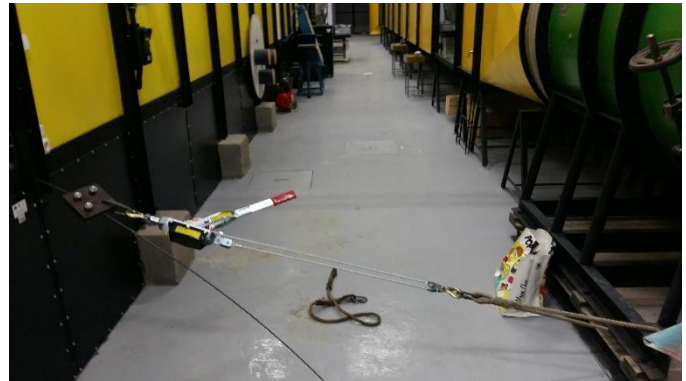


Figure III-2 – Active anchorage

In order to increase the effects of the wind and at the same time to ensure a constant section with as few irregularities as possible, the cable was inserted into a round sponge sheath with an outer diameter of 50mm and a thickness of 8mm ($\varnothing 50 \times 8$), which was later filled with polyurethane foam to ensure the fixing of the cable with minimal influences on its stiffness. The weight per meter of the assembly consisting of cable, sheath and foam is about 224 g / m.

The cable was instrumented with three three-dimensional accelerometers mounted at a quarter of the length from the top, in the middle, and a quarter from the bottom. Hereinafter, they will be named in the order specified "Acc - top", "Acc - Center", respectively "Acc - Down".

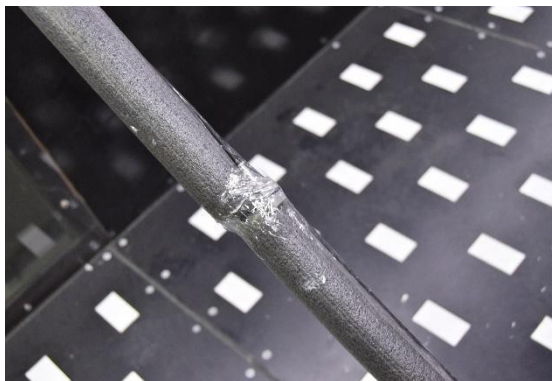


Figure III-3 – Instrumenting the cable with accelerometers



Figure III-4 – Scale model (tensioned cable)

The accelerometers used have the ability to simultaneously record accelerations in 3 orthogonal directions with a frequency of 1200Hz (a recording at 0.00083s). They were mounted so that the "X" direction coincided with the cable direction, the "Z" direction coincided with the wind direction (horizontal, perpendicular to the cable), and the "Y" direction was in the vertical plane, perpendicular to the cable. Because the accelerometers are square in shape and can alter the aerodynamic profile of the sheath, they were fixed behind the cable in the

direction of the wind and covered with tape to mimic the aerodynamic profile of the sheath as accurately as possible.

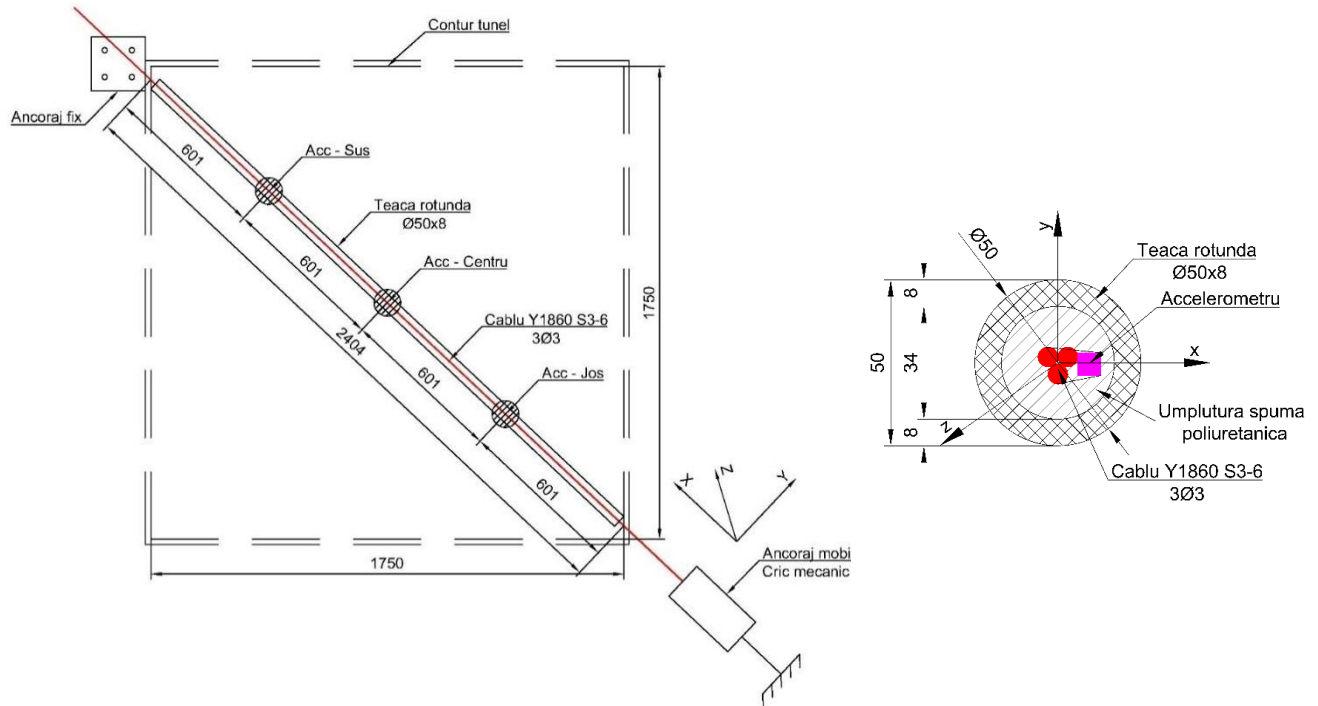


Figure III-5 – Sketch with the scale model and section of the cable

Three test scenarios were first proposed to determine the dynamic response of the cable, with its internal tension variation from case to case, respectively with 118.95N, 288.27N and 655.10N. For each scenario, several tests were performed, with the measurement of three-dimensional accelerations in the 3 positions at intervals of 60s each and with the recording frequency of 1200Hz, as follows:

- Free vibrations - measuring the behavior under free vibrations, without the action of the wind; it was made by introducing an impulse, suddenly released by means of a deformation imposed in the middle of the cable of 10 cm in a direction perpendicular to it in a vertical plane (Y direction); The aim was to determine the fundamental period of vibration, the level of damping and, indirectly, the determination of the actual induced tension.
- 0-5m/s speed - tracking the behavior under the action of the wind in a direction perpendicular to it, with the variation of the air speed from 0 to 5m/s
- 5m/s speed - tracking the behavior under the action of the wind, with a constant air speed of 5m/s
- 10m/s speed - tracking the behavior under the action of the wind, with a constant air speed of 10m/s
- 12m/s speed - tracking the behavior under the action of the wind, with a constant air speed of 12m/s
- 15m/s speed - tracking the behavior under the action of the wind, with a constant air speed of 15m/s
- Decreasing speed - tracking the behavior under the action of the wind, with the decreasing speed from 15m/s to stopping

Following all measurements and a preliminary analysis of the results, the strongest oscillations occurred on the cable tensioned at 655.10N at a wind speed of 5m/s. In order to simulate the most unfavorable situation, it was decided to carry out another set of tests with wind speeds around 3m/s.

Due to the large volume of data, only a few situations are presented in this paper, at intervals of maximum 10s for each measurement. The results were processed by applying baseline corrections and bandwidth filters with frequencies between 0.1Hz and 100Hz. Fourier spectra and power spectra were calculated to show the results in the frequency range as well.

III.2.3.2. Results

Given the large amount of data recorder in the experiment, only the scenario considered to be most edifying will be presented below.

I.1.1.1.1. Wind vibrations with a speed of 3m/s on the tensioned cable at $F = 655.10N$

The recordings corresponding to this analysis were made over a constant air speed range of 3m / s. Below is a 6s interval characteristic of oscillation stability.

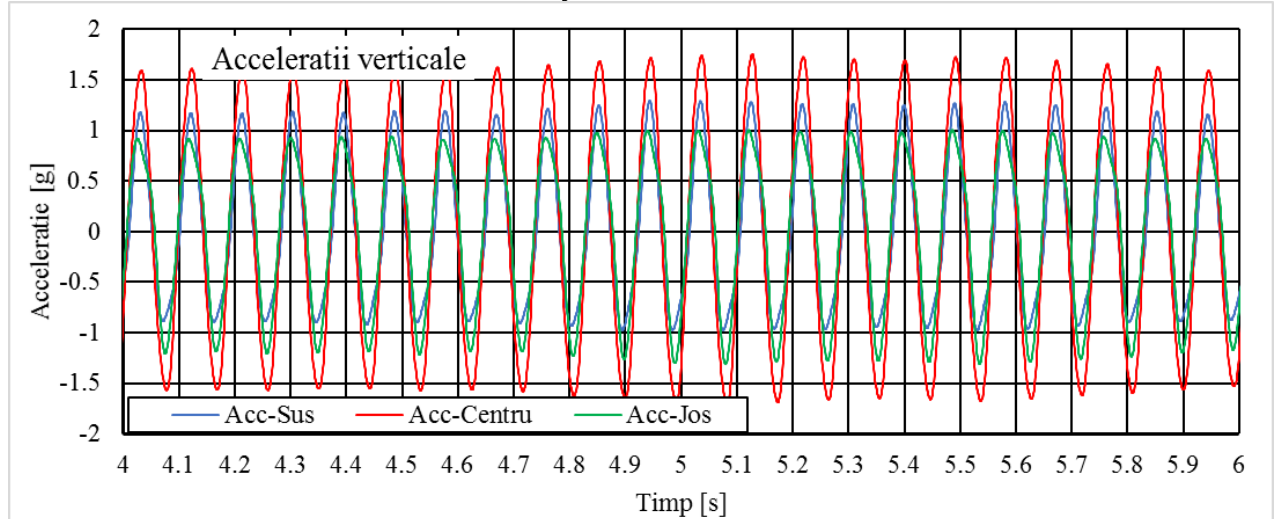


Figure III-6 – Accela  ii verticale  n cablu pe interval de timp restr ns

The vertical response shows a cyclic motion, with a stable period and correlation between all measuring points. The vibration amplitudes are approximately equal from one cycle to another, around 1.6-1.7g for the midpoint and 1.2-1.3g for the edges.

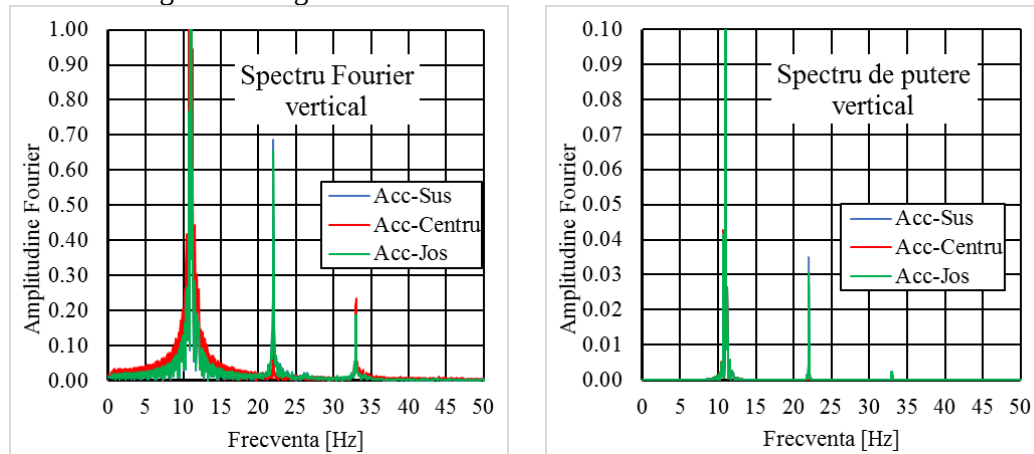


Figure III-7 – Fourier and power spectra

III.2.3.3. Simulation of vibrations recorded by numerical methods

One of the main objectives of this study is to use the data obtained from the measurements in order to achieve a sufficiently accurate methodology for calculating the vibrations of the cables, which can also be applied to real-scale structures.

As so, it was proposed to break down the specific phenomena into 2 numerical models as follows:

- An aeroelastic model with finite elements in the field of fluid mechanics, with the purpose of estimating the movement of air around the cable, the level and the variation in time of the forces acting on it under the wind;
- A model with finite elements in the field of structural mechanics, with the purpose of estimating the behavior of the cable under the external actions taken from the previous model;

I.1.1.1.2. Aeroelastic finite element model in the field of fluid mechanics

The airflow around the cable and its effects were estimated in a two-dimensional finite element model. The cable was modeled like a stationary wall, in the shape of a circle with a diameter of 5 cm, and the analysis area (air mass) was considered rectangular, so that both in the horizontal direction in front of the wall and in the vertical direction above and below, there should be spaces of at least 3 diameters of the cable sheath, and behind the wall, in a horizontal direction at least 5 diameters, in order to better follow the phenomenon of vortex shedding.

The contour conditions were introduced as follows:

- Left edge - intake
- Right edge - evacuation
- Top, bottom edges - lines of symmetry (frictionless surfaces)
- Cable - wall with friction on the contour

The mathematical model consists of 21350 flat finite elements of quadrilateral type with 8 nodes (4 at the corners and 4 at the middle of the sides), resulting in a total of 21655 nodes. The discretization was done by making the dimensions of the finished elements smaller around the cable surface and larger towards the edges. As so, dimensions of the sides between 0.5mm and 5cm resulted.

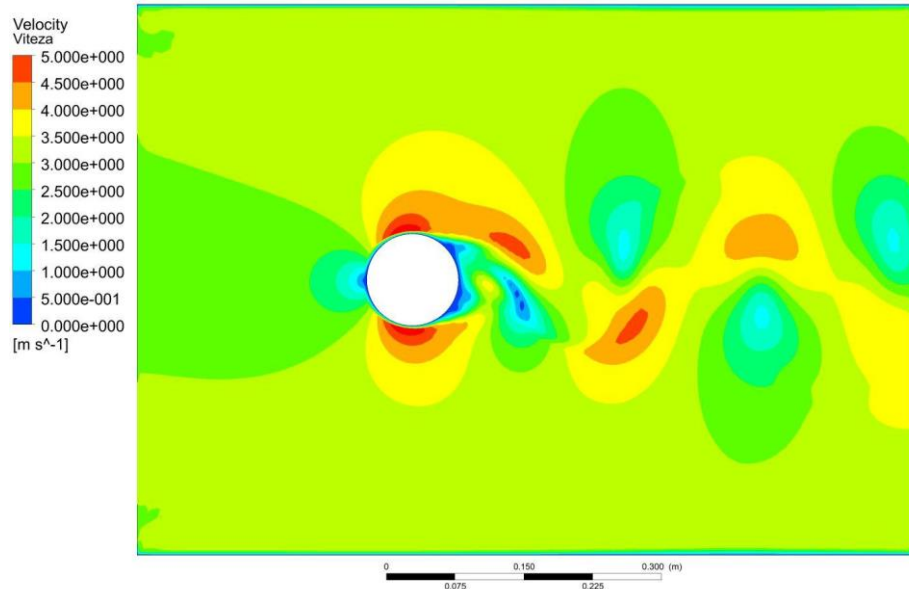


Figure III-8 – Total speed contour

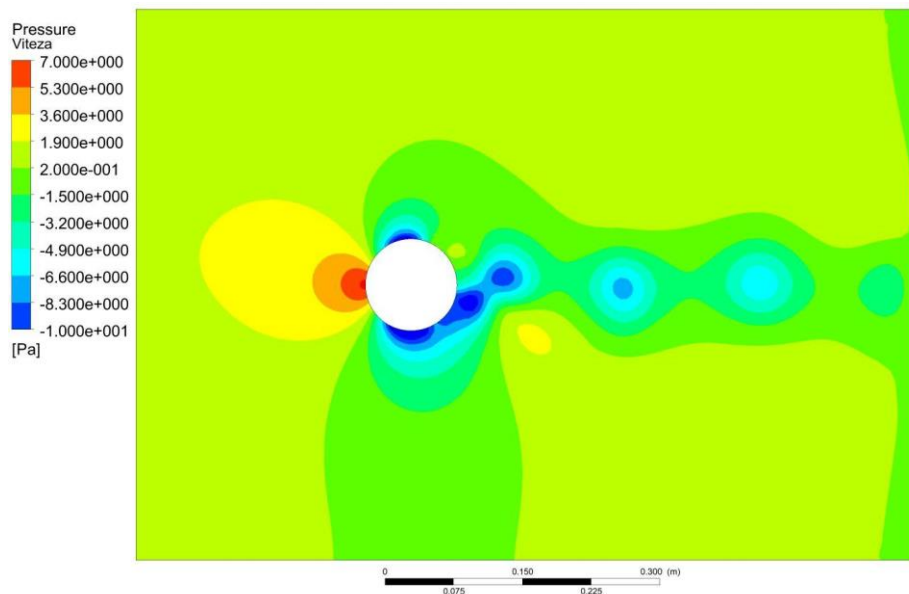


Figure III-9 – Pressure contour

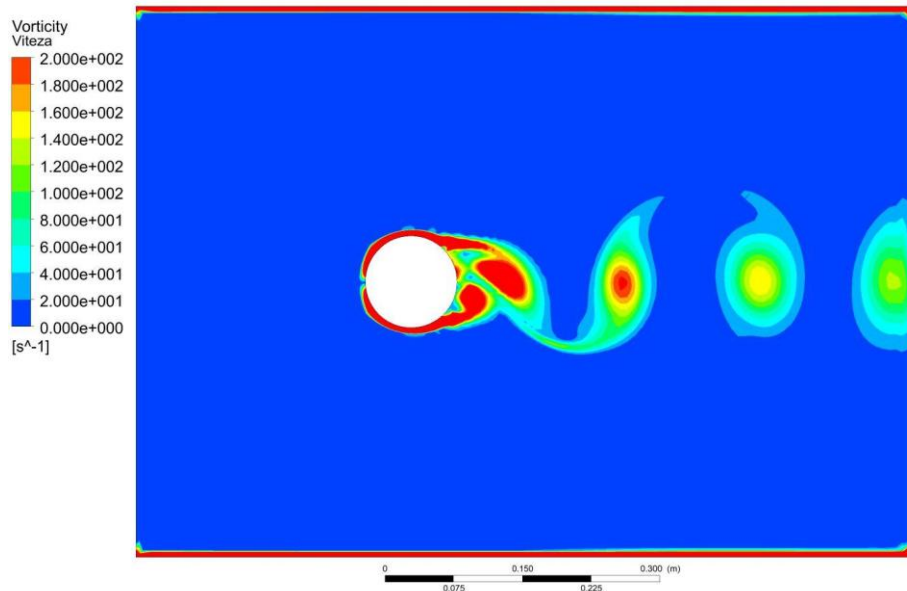


Figure III-10 – Vorticity contours

The main purpose of the model was to track the effect of airflow around the cable. For this reason, the time variations of the aeroelastic forces acting on the cable were monitored in horizontal and vertical direction. Below are graphs with the variation in time of these forces depending on the base air speed.

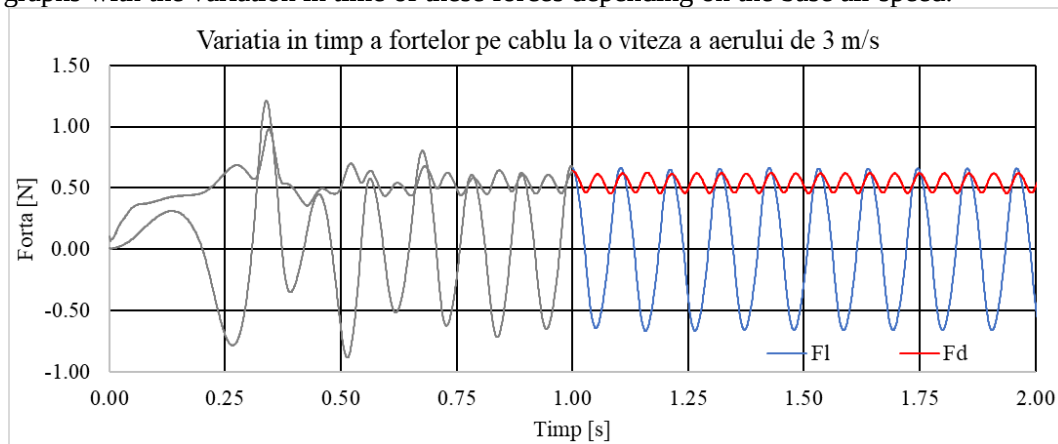


Figure III-11 – Time variations of the forces acting on the cable at wind speeds of 3m/s

The chart presented above shows that the wind-induced forces on the cable have comparable maximum values on the 2 directions of movement, but their variations over time are different. On the wind direction (z-z) it is found that the forces variations are of a cyclical nature with stable period and amplitudes, with the reference line of oscillations around an average value, corresponding to a drag coefficient of 0.8, which corresponds to corresponding literature. In the perpendicular direction to the wind, it was also found that the forces vary cyclically with stable period and amplitudes, but with the line of symmetry around 0, which leads to much higher amplitudes of oscillations. Also, the period of vertical oscillations is about twice that of the horizontal.

Basically, the phenomenon can be reduced to a force along the constant wind direction over which a force with a small amplitude sinusoidal shape overlap, and on a vertical direction with a force with a sinusoidal variation, with a much larger amplitude

I.1.1.1.3. Finite element model in the field of structural mechanics

The studied cable was modeled by means of bar-type finite elements 5 cm long (49 in total), fixed at the ends. For axial bending stiffness, geometric features equivalent to 3 times those of a circular bar with a diameter of 3mm (3Ø3) were introduced. The bending stiffness of the stays can usually be neglected, but in this case it is significant in relation to the length of the cable.

The last scenario, with the highest level of prestressing, estimated at 655N, was chosen for comparison. The introduction of the internal tension in the calculation model was done by a specific deformation imposed by $-1,584 \times 10^{-4}$, applied along the entire length of the cable in a nonlinear analysis with P-delta effects. All subsequent analyses will take start from the state of deformations and stiffness at the end of this load case.

A first check of the accuracy of the calculation model was made by comparing the first 2 eigen modes of vibration, as follows:

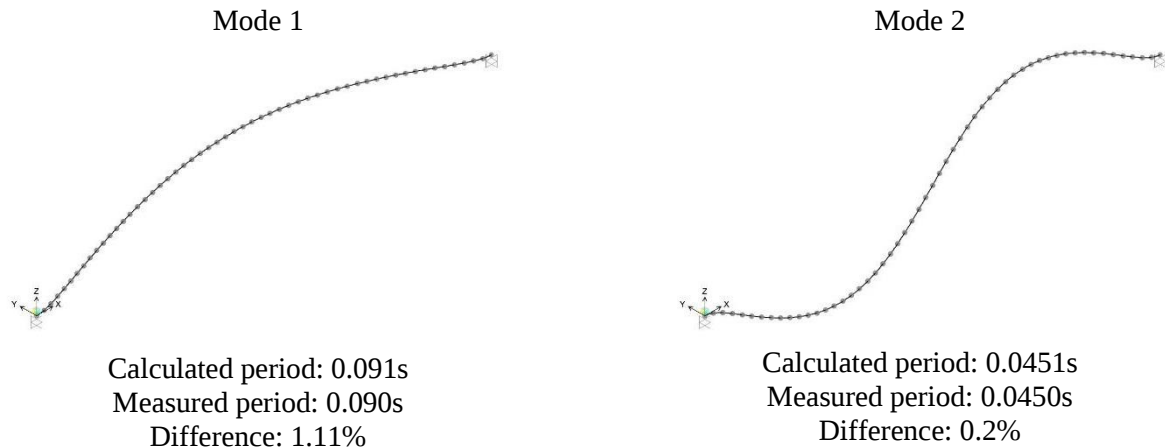


Figure III-12 – Comparison of the calculated and measured modes of vibration

As it can be seen, the dynamic characteristics of the modeled cable compared to the measured one are very close, with errors of less than 1.5%, small enough to be neglected for the case study.

The following will compare the results obtained in 3 distinct cases, as follows:

- Behavior of the cable under free vibration
- Behavior of the cable under constant wind speed of 3m/s
- Behavior of the cable under constant wind speed of 5m/s

The comparison criterion was considered the acceleration in the y-y direction in the middle of the cable (perpendicular to its direction and that of the wind). Nonlinear time-history analyses with P-delta effects and large displacements were performed, using an explicit integration method (HHT, $\gamma = 0.5$, $\beta = 0.25$, $\alpha = 0$). Internal damping, which plays a key role in this type of analysis, was considered by the Rayleigh method. The coefficients α and β related to the method were established by a parametric variation with multiple analyses, so as to obtain results as close as possible to those measured.

1.1.2. Comparison between the recorded results and those obtained by calculation

To simulate the response of the cable under the wind with a constant speed of 3m/s, the forces and their time per unit length and their variations, determined in the previous chapter were considered, and applied as forces evenly distributed on the cable, along its entire length except at the ends, where for about 30 cm it was considered that the walls of the tunnel influence the shedding of Von-Karman vortices (considered to be mainly responsible for the variations of the forces) by a substantial amount.

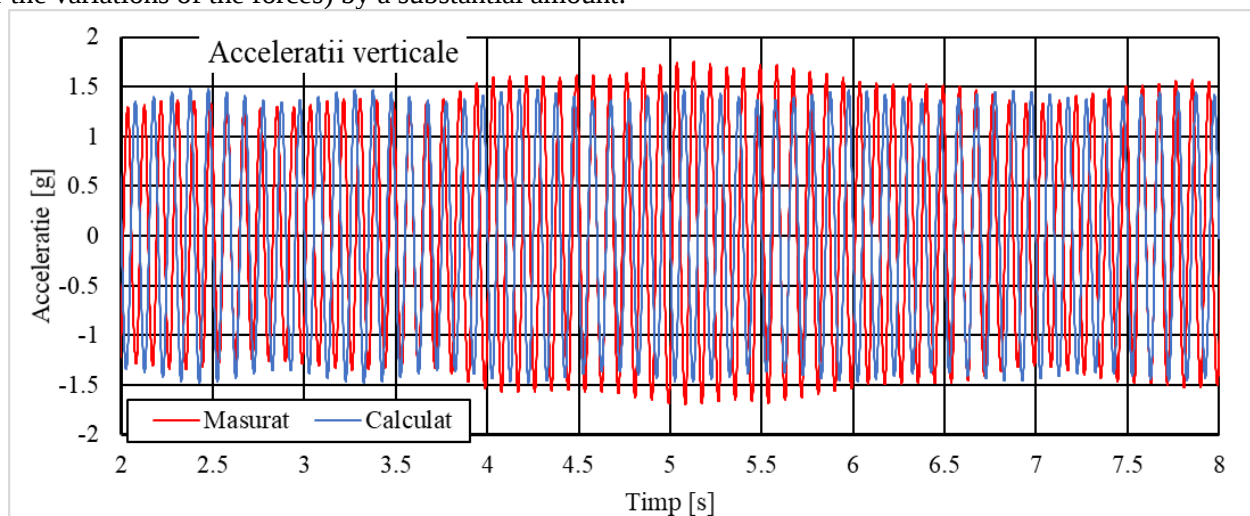


Figure III-13 – Comparison between measured and calculated cable vertical accelerations

The cable response was calculated with an integration time of 0.001s, for 36s, of which the above graph shows a range of only 4s of accelerations calculated in the Y direction in the middle of the cable, overlapping with those measured. It took about 28s to reach stable oscillations of the cable, which is why only the results at the end of the analysis are presented.

Over the presented range, the compared accelerations are almost identical, both in terms of amplitudes and laws of variation. Maximum values alternate between calculated and measured accelerations, but with small differences of less than 5%. The numerical model shows harmonic motion with stable cycles, periods of 0.09s and repetitive intervals of small increases / decreases in amplitudes of approximately 1.00s. The measured accelerations are similar, but with a very low degree of instability, observed by irregular intervals of increases / decreases in amplitudes. These can be considered normal when taking into account the large number of phenomena that occur simultaneously and the influences of contour conditions (vibration of tunnel walls, imperfect anchors, etc.).

CHAPTER II – CASE STUDY - VLADIVOSTOK CABLE STAYED BRIDGE

II.1. Purpose and objectives of the study

The case study will condier the example of the Russky Bridge in Vladivostok, which holds the record for the largest main span of a cable stayed bridge and thus, the longest cables, at the time of writing. Its composition has been presented in detail, together with the cable vibration damping systems adopted, in the previous chapters (See II.5.3).

The main purpose of the study is to estimate the dynamic response of the cables of the analyzed bridge with the very large span (1104m) in different design situations and to determine the influence of damping systems on them.

One of the main objectives is to determine the dynamic response of the cables in the absence of vibration damping devices, in order to highlight potential vulnerabilities in the structure generated by uncontrolled amplified oscillations.

Another objective is to determine the dynamic response of the cables in the situation where conventional external viscous dampers are attached to their base, at the level of the superstructure. These results are needed to compare the two behaviors and highlight the improvements made by the damping devices.

The third objective is to study the possibility of using a new damping solution, by applying viscous dampers fixed between adjacent cables, and not fixed to the deck as is the conventional solution for this type of structures. The aim is to determine the dynamic response in this situation and to compare with the other two. Studies will be performed to determine both the optimal characteristics of the devices (viscosity, speed exponent, elastic stiffness, etc.) and the optimal position of the shock absorbers along the cable. Multiple shock absorber arrangement schemes will also be studied to determine the most effective scenario.

II.2. Calculation approach

II.2.1. Structural elements considered and their modeling

Finite element numerical models were used to estimate the response of the cables by calculations. Due to the complexity of the analyses and the large volume of computing power required, it was decided not to model the entire structure, but instead to isolate and only model the longest 6 cables. These were modeled as double-articulated individual bars. Each stay was modeled with 200 finite bar elements with 2 nodes at each end, each with 6 degrees of freedom (3 translations and 3 rotations). Their anchorages were arranged at horizontally at distances of 20.00m at the level of the deck, respectively vertically at 5.50m at the level of the pillar. The longest cable considered is 604.00m in length, and the shortest one is 486.00m long. Further details on their arrangement and their geometric features can be found in Figure IV-1.

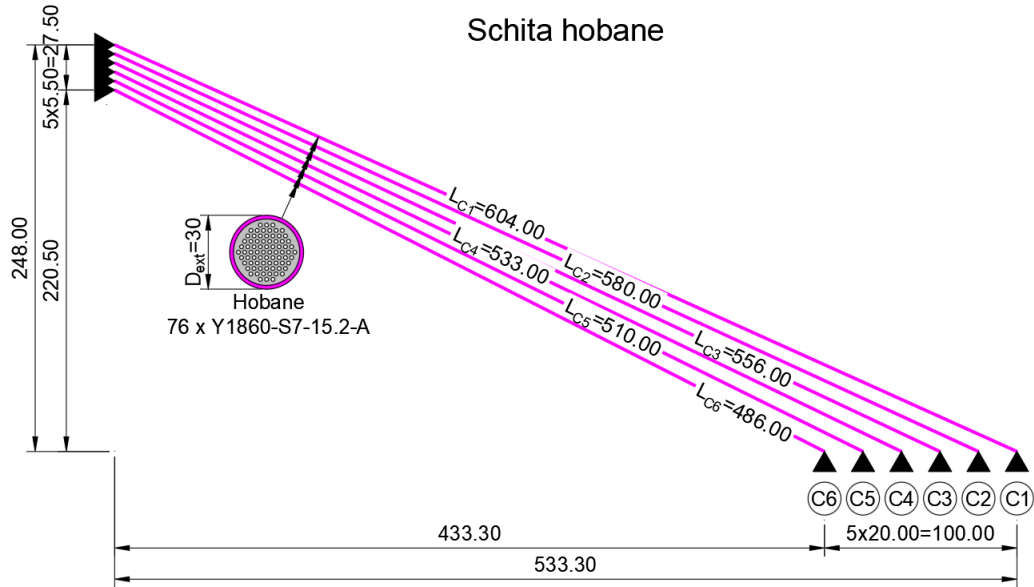


Figure IV-1 – Sketch of the modeled cable stays

Cables consisting of 76 parallel strands of type Y1860-S7-15.2-A (SREN1993-11), inserted in HDPE sheaths with an outer diameter of 30 cm were considered. For a higher accuracy of the results, especially in the anchorage areas, a bending stiffness of the stays was considered, as the equivalent of $76 \times 7 = 532$ wires with a diameter of 5mm (Felix Weber H. D., 2014) and the modulus of elasticity $E_P = 195 \text{ GPa}$ (SREN1993- 11).

II.2.2. Exterior excitation

Based on the wind tunnel studies presented in the previous chapters, the external excitation was considered as a uniform distributed time varying force acting on the entire length of the cables, perpendicular to them in the vertical plane. To simplify the calculations, the coupling of the wind action on the 3 orthogonal directions or the variation of its intensity on the height was not considered.

Schita excitatie exterioara

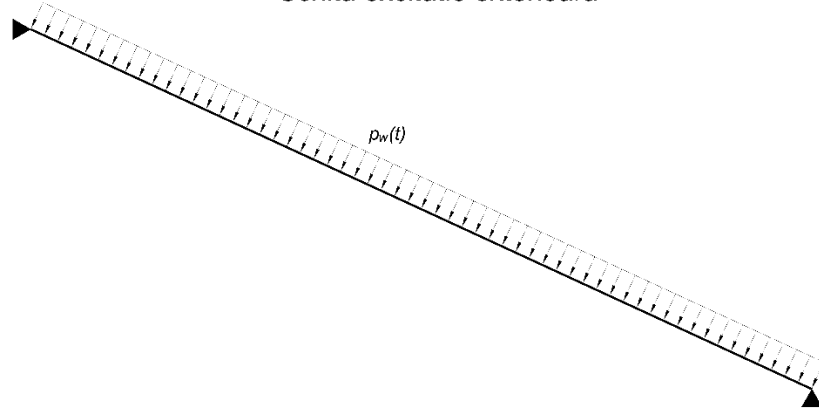


Figure IV-2 – Sketch of the exterior excitation modeled as a uniform distributed force

$$p_w(t) = c_l \cdot \frac{\rho \cdot v_w(t)^2}{2} \cdot \sin\left(\frac{2 \cdot t \cdot \pi}{T}\right) \quad (\text{IV.1})$$

Unde: $c_l = 0.8$ coeficient de liftare (Chanson H. , 2009)

$\rho = 1.25 \text{ kg/m}^3$ (SREN1991-1-4)

$T = 1.12 \text{ s}$ vortex shedding period (II.2.3)

$v_w(t)$ – the time variation of the wind speed determined with the help of the application developed by the University of Notre Dame, considering the exposure class “A” according to ASCE 7-98 and a base wind speed of 12m/s - <http://windsim.ce.nd.edu> / (Kwon, 2006); The function obtained can be seen in the following graph

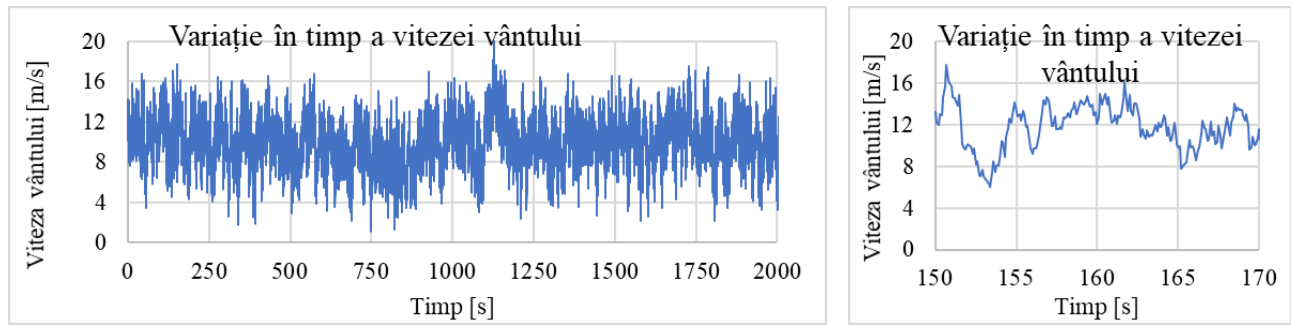


Figure IV-3 – Wind speed time variation entire considered interval of 2000s, respectively in detail over a reduced section of 20s

Based on the speed variation presented above, the following time variation of the force acting on the cables is obtained:

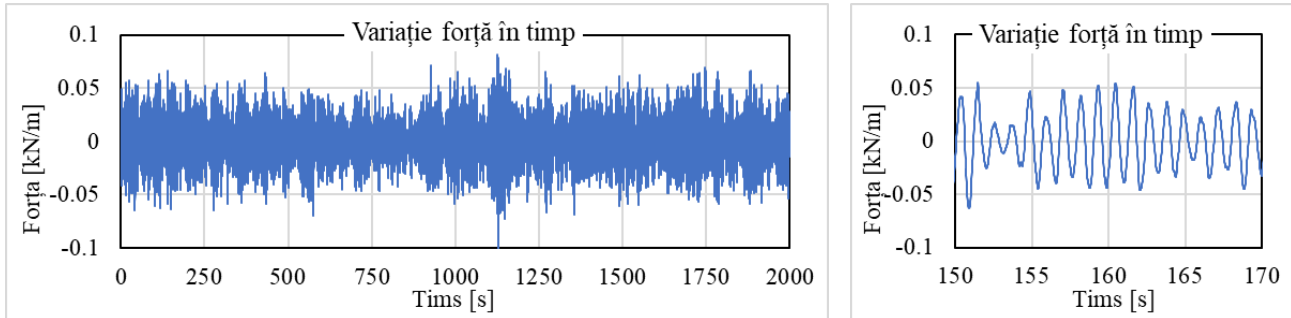


Figure IV-4 – Time variation of the external force

II.2.3. Proposed devices for cable vibration damping and considered parameters

Out of the wide range of existing vibration damping devices for cables, only viscous external dampers were considered in this study. Regardless of their position, whether they are fixed at the bottom between the cables and the deck, or they are fixed between adjacent cables, the mathematical formulation of the force generated by the device is:

$$F(t) = K \cdot \gamma_d(t) + C \cdot v_d(t)^\alpha \quad (IV.2)$$

Where the following parameters were considered: elastic stiffness $K = 10000 \text{ kN/m}$, velocity coefficient $\alpha = 1$, and viscosity coefficient c determined by a parametric study performed in order to find the optimal value, starting from the recommendation to determine the optimum viscosity coefficient according to (Wang X. Y., 2004):

$$c_i^{opt} = \frac{1}{i \cdot \pi \cdot \frac{x_d}{L}} \cdot \sqrt{T \cdot m}, \quad (IV.3)$$

Where: $i = var [1 \dots 5]$, x_d varies depending on the situation analyzed, $L = 500m$ (average length of the considered cable bundle), $T = 6500kN$, $m = 85.509 \text{ kg/m}$

II.3. Dynamic response of the undamped cables

In order to establish a reference and have a basis for comparison, the dynamic response of the undamped cables was initially calculated. This analysis also aims to highlight critical areas with potential vulnerabilities and possible degradation caused by uncontrolled vibrations with high amplitudes.

The dynamic response obtained was similar for all 6 cables, with the highest amplitudes in the C6 cable, but with values similar to these obtained in the other cables. For later comparisons, in the following there is a graph with the displacements in time of the point in the middle of the C4 cable.

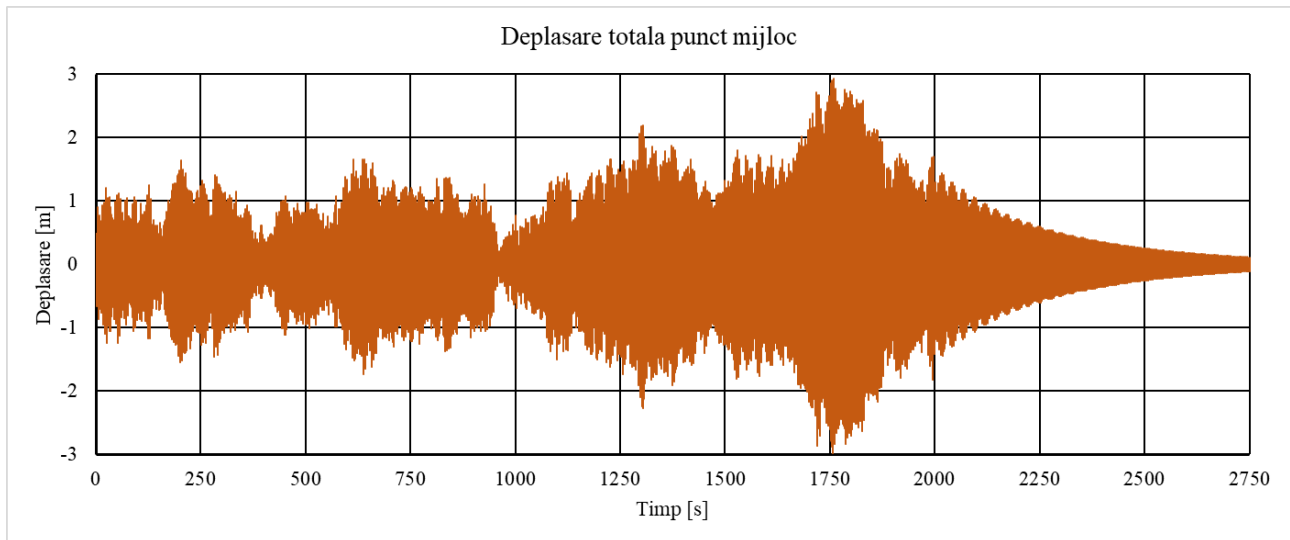


Figure IV-5 –Graph with the displacement of point in the middle of the cable over time

By analyzing the previous graph, we notice first of all the appearance of resonance and beats phenomena. From the first amplitudes which are about 0.50m, the cable reaches amplitudes of up to 6.00m. In the interval of forced vibrations, the movement seems to describe stable oscillations over time, but with varying amplitudes. Their average values are about 2.55m. It is observed that amplitudes reach maximum values by the phenomenon of beats at time intervals of approximately 450s, around the seconds 205, 650, 1303, respectively 1750.

From 2000th second, the external excitation stops, and the cables enter free vibrations, a change that can also be seen in the graph. The movement becomes more stable with decreasing cycles and macrocycles, respectively. The very low internal damping means that the vibrations do not completely stop even after the relatively long time interval analyzed of 750s (over 12 minutes).

For a more detailed description, below are graphs with the oscillation of the cable at short intervals over the range of forced vibrations, respectively the one with free vibrations.

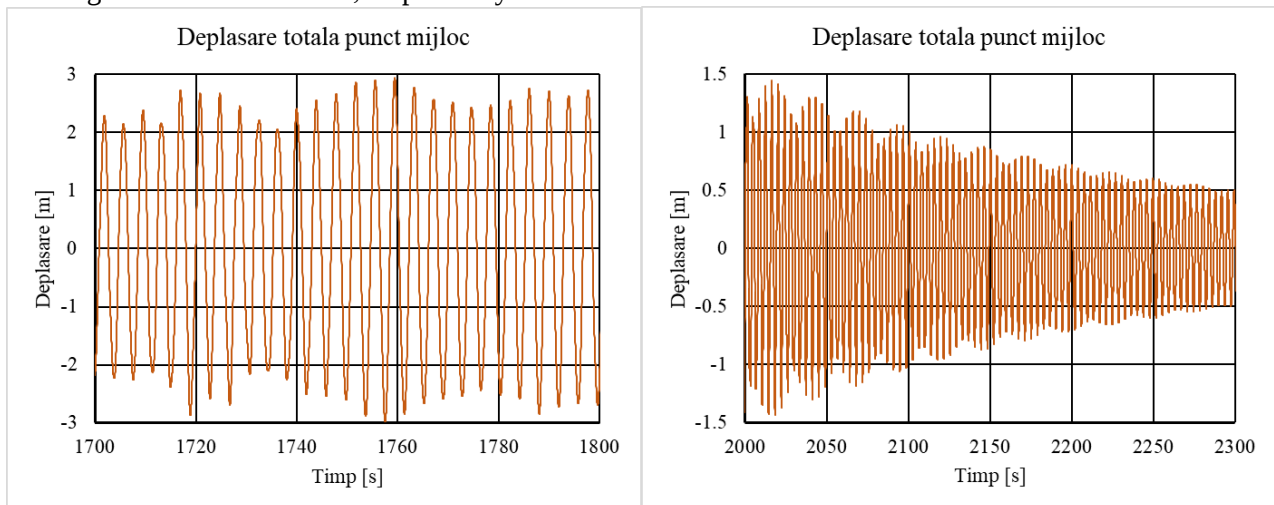


Figure IV-6 – Graphs over short intervals with the movement of the middle of the cable over time

From the previous graphs, the beats phenomenon is much better highlighted, being able to show the cycles with a period of about 4.00s and the wave described by the maximum values of the amplitudes with a period of more than about 40s. There are both intervals of increasing amplitudes and decreasing them, depending on the overlap of their own vibration frequencies. They are also present in the range of free vibrations, but with a reduction of the period of the beats phenomenon to about 25 s and a reduction of the amplitudes of its wave as the vibrations decrease.

II.4. Influence of viscous external dampers attached to the base of the deck on the dynamic response of the cables

Since the 6 studied cables have similar geometric properties and dynamic characteristics, taking into account the fact that in the study hypotheses they are isolated and work uncoupled, without influencing each other, only 2 stays were chosen for analysis (C2 and C3), in order to reduce the required computing power. The dampers were modeled by means of specialized finite elements of the viscous damper type. These elements each have two

nodes and six degrees of freedom, just like a simple bar, but with non-linear behavior of hysteretic type, dependent on speed and not deformation. These elements allow the definition of the viscosity coefficient, the speed exponent, as well as all other specific parameters.

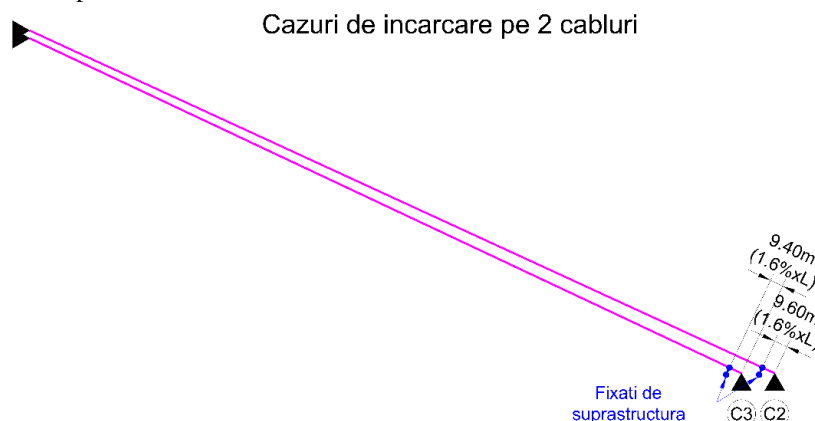


Figure IV-7 – Scheme with arrangement of the viscous dampers at deck level

A height of approximately 5.50m was considered from the plane where the cable anchors are found to the level of contact with the dampers, resulting in distances along the cables between the anchors and shock absorbers of 9.40m for the C3 hob, respectively 9.60m for the C2, representing approx. 1.6% of their length.

The parameters of the viscous dampers have been specified in the previous chapters (Chapter II.3.4.3), except for the viscous coefficient, for which the study contains several values, starting from the optimal value resulting from the literature for the proposed position and for mode of vibration 1. Its value has been varied in order to obtain the highest possible efficiency of the device. The following table and graph show the energy dissipated by a damper according to the value of the viscosity coefficient.

Results table			
Viscous coefficient c	Percentage of theoretic c optimal	$F_{\max}$ damper	Dissipated energy
[kN x s / m]	[%]	[kN]	[kJ]
0			0
40	10.00%	10	509
133.33333	33.33%	18	599
200	50.00%	21	567.5
400	100.00%	25	430
800	200%	38	230

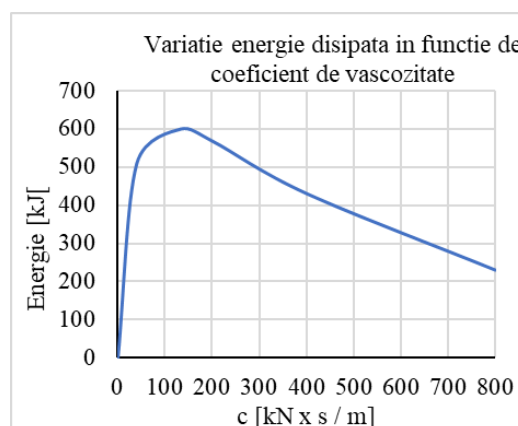


Figure IV-8 – Influence of the viscosity on the cable response

The maximum forces in the dampers were determined on the basis of the corresponding hysteresis graphs, similar to those in. Figure IV-8. The dissipated energy was calculated by integrating the mechanical work done by the damper over the studied time interval.

As can be seen from the dissipated energy, the most efficient value for the viscosity coefficient was 133.33 kN x s / m, corresponding to 33% of the estimated theoretical optimum value for mode 1. This difference can be explained by the large influence of the higher vibration modes, in detriment to the fundamental one. The graph shows the narrow range of values of the viscous coefficient for which a satisfactory efficiency level is obtained. From the graph in Figure IV-8 it can be seen how the amount of energy dissipated over the studied time interval increases suddenly with the viscosity, has a narrow interval of maximum efficiency, after which it decreases asymptotically to about 40% of the maximum value. These results are similar to other works in technical literature (Habib Tabatabai, 2003).

Below are graphs with the displacement of the point in the middle of the cable, respectively with the variation of the axial force in the anchorage, by comparison with the scenario without dampers for the case with the highest efficiency.

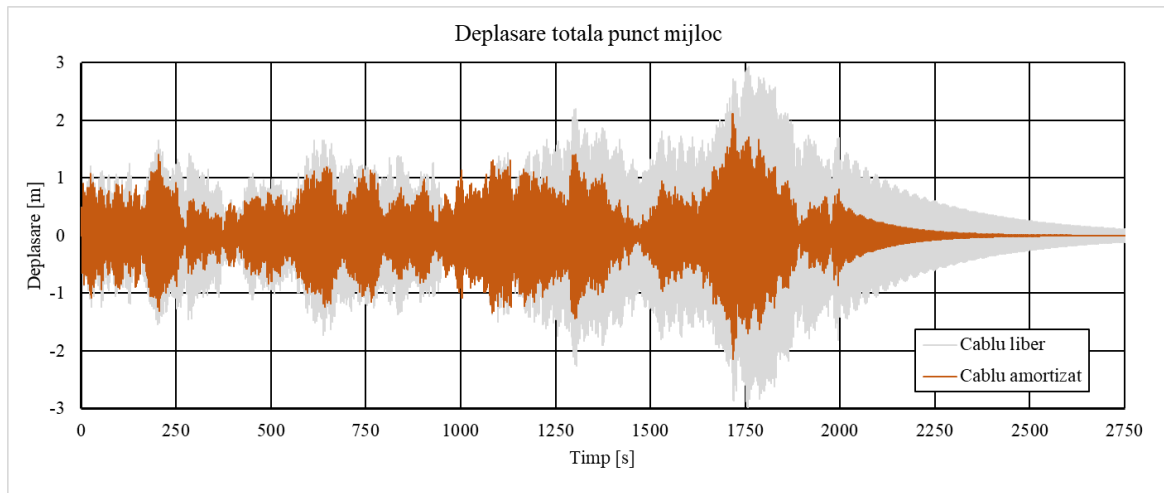


Figure IV-9 –Graph of displacement the middle of the cable over time

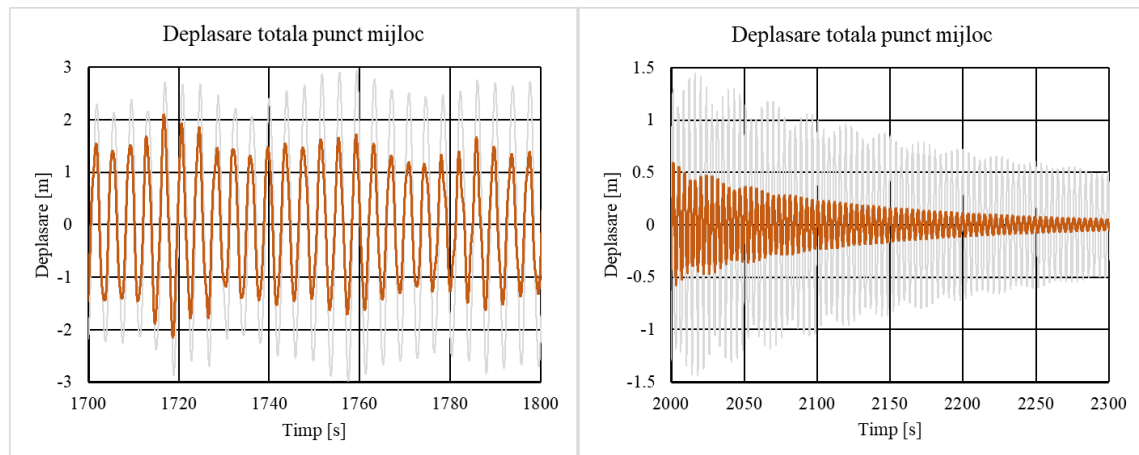


Figure IV-10 – Graphs at short intervals with the displacement of the middle of the cable over time

In terms of cable movements, the amplitudes of the oscillations are about 35% lower. The maximum amplitudes of the movements are around 2.20m, with a maximum of 4.00m. In general, the variation of the movement is similar to that of the undamped situation, but with lower values of the amplitudes. Resonance and beats phenomena, with similar cycles and macro cycles, are still present.

The chosen damping system has a small influence at lower amplitudes, but an increased efficiency at higher displacements. Certain intervals are observed, especially at low amplitudes, in which the variation of the displacement described in the case studied and the reference one is similar, with insignificant differences (example between 0 and 50 seconds, or between 900 and 1000).

In the range of free vibrations (after the second 2000), after only 250s, the amplitudes of the vibrations are reduced to an acceptable level, unlike the base scenario in which this does not happen even after 750s.

In order to highlight how the damping device dissipates energy, the hysteretic curve described by the viscous damper in the maximum efficiency scenario is presented below.

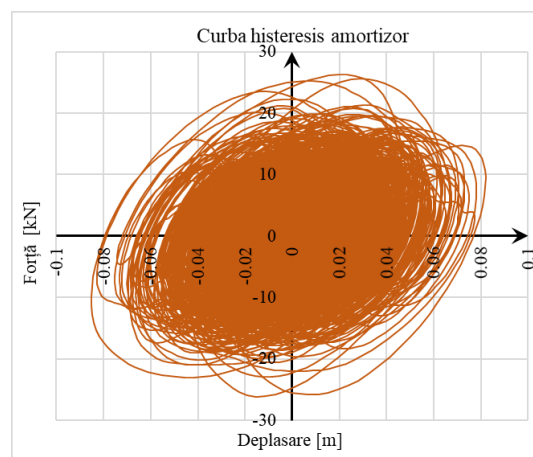


Figure IV-11 – Hysteretic curve of the damper

The total stroke of the piston is about 16 cm, and the shear force induced in the cable is less than 30kN, which is acceptable to prevent degradation of the stays elements due to local effects.

The analysis of the oscillations showed a 200% increase in damping, with a critical damping coefficient of 1.04%, respectively a value of the Scruton number of $Sc = 7,751$. Compared to the recommendations of the technical literature, this value, although it is double that of the base case, is still not satisfactory.

II.5. Influence of viscous external dampers fixed between adjacent cables on their dynamic response

II.5.1. The influence of the position of viscous dampers on 2 isolated stays

One of the most important parameters that needs to be determined for the efficiency of an external damping device is its position along the cable. In order to determine the optimal characteristics for the external damping devices fixed between adjacent cables, initially only the two cables from the previous chapter were considered, which enables the installation and analysis of a single device. A parametric study was performed with the variation of the value of the viscous coefficient, starting from the basic value theoretically determined for vibration mode 1 relative to the position of the device along the cable, for which 7 situations were also considered as follows: at 50% length, 33%, 25%, 20%, 15%, 10% and 5%, respectively.

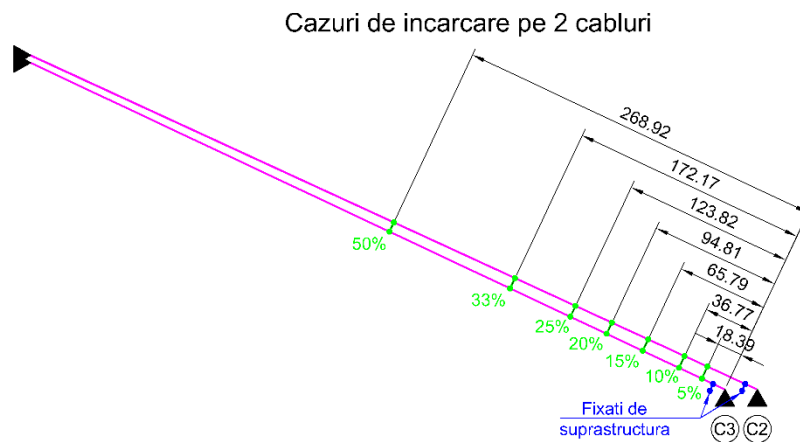


Figure IV-12 –Positions considered for viscous dampers between cables

Related to this chapter, a number of 49 analyses were performed with the variation of the viscosity coefficient for all the 7 loading positions, as a result of which the following graphs with values of dissipated energy were obtained:

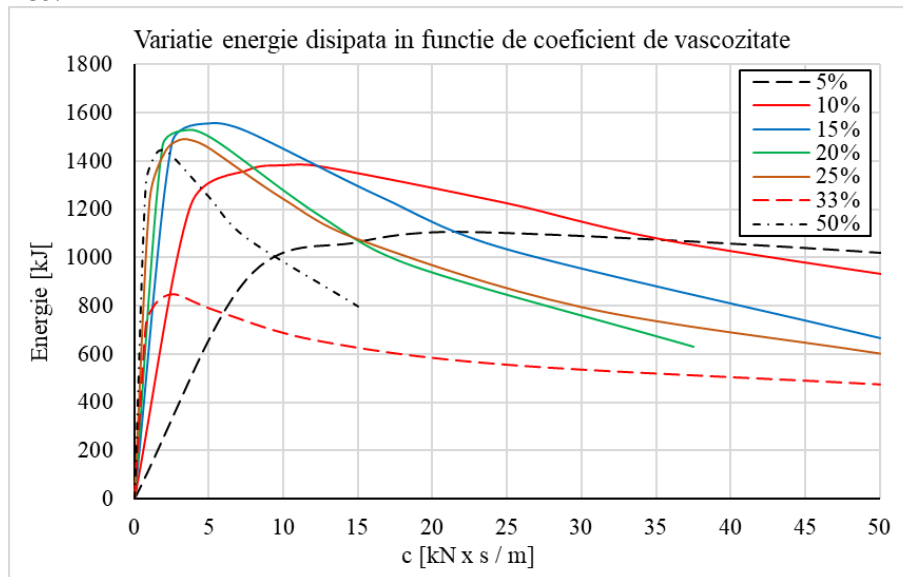


Figure IV-13 –Variation of energy dissipated by the damper depending on position and viscosity

The shape of the graphs is similar to that of the dampers attached to the superstructure. As the distance between the anchor and the position of the device increases, the range of values for the viscous coefficient for which the amount of energy dissipated is high also increases. Acceptable values are obtained for damper positions

greater than 10% along the cable, except for the case where the relative position was at 33%, for which the obtained response was unsatisfactory.

The percentage of critical damping and the Scruton number were calculated for each position of the damper. These are presented in the following table:

Table 1 – Analysis results for a single damper attached between adjacent cables

Damper position along the cable	Optimal theoretical viscosity		Critical damping percentage ξ	Scruton number
	Viscous coefficient c	Percentage of optimal c		
	[kN x s / m]	[%]	[%]	[]
1.60% (Fixed to the deck)	133.33	33.33%	1.04%	7.751
5%	22.5	15.00%	0.81%	6.040
10%	10	13.33%	1.08%	8.008
15%	5	10.00%	1.44%	8.458
20%	3.75	10.00%	1.53%	9.144
25%	3	10.00%	1.40%	8.148
33%	2.5	10.00%	0.89%	6.630
50%	1.5	10.00%	1.44%	8.490

According to the table, the positions where the device is most effective are between 15% -25% of the length of the cable, with percentages of the corresponding critical damping of over 1.40%, almost 3 times more than the situation without dampers.

Graphs with the displacement of the point in the middle of the cable for the case with the highest efficiency, respectively with the variation of the axial force in the anchorage compared to the scenario without dampers are presented below.

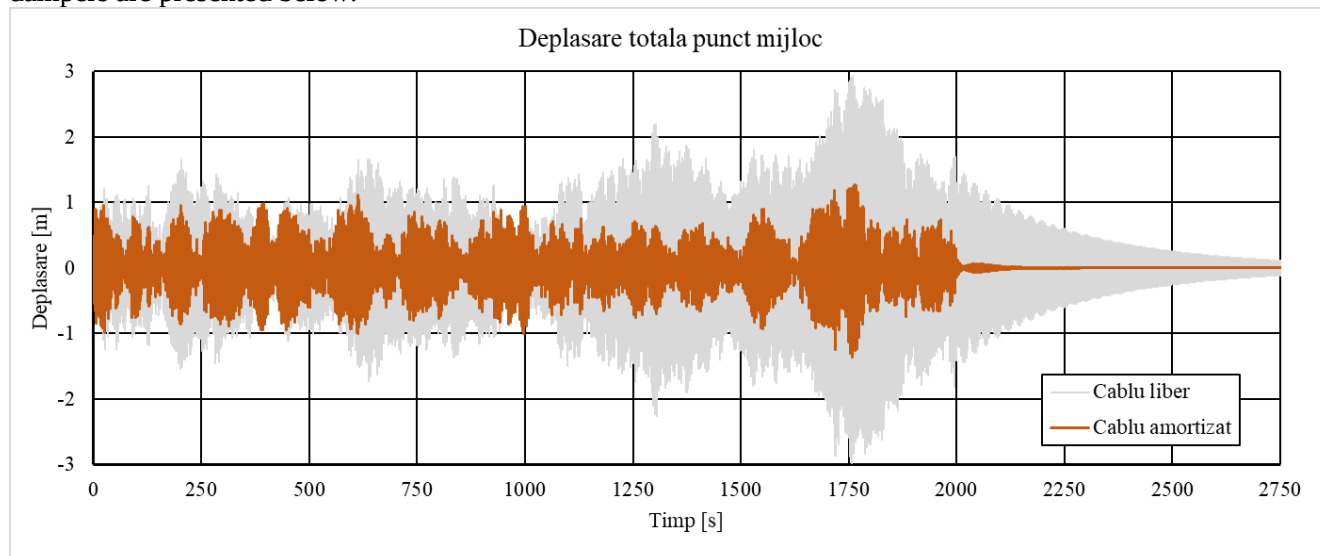


Figure IV-14 – Graph with the displacement of the middle of the cable over time

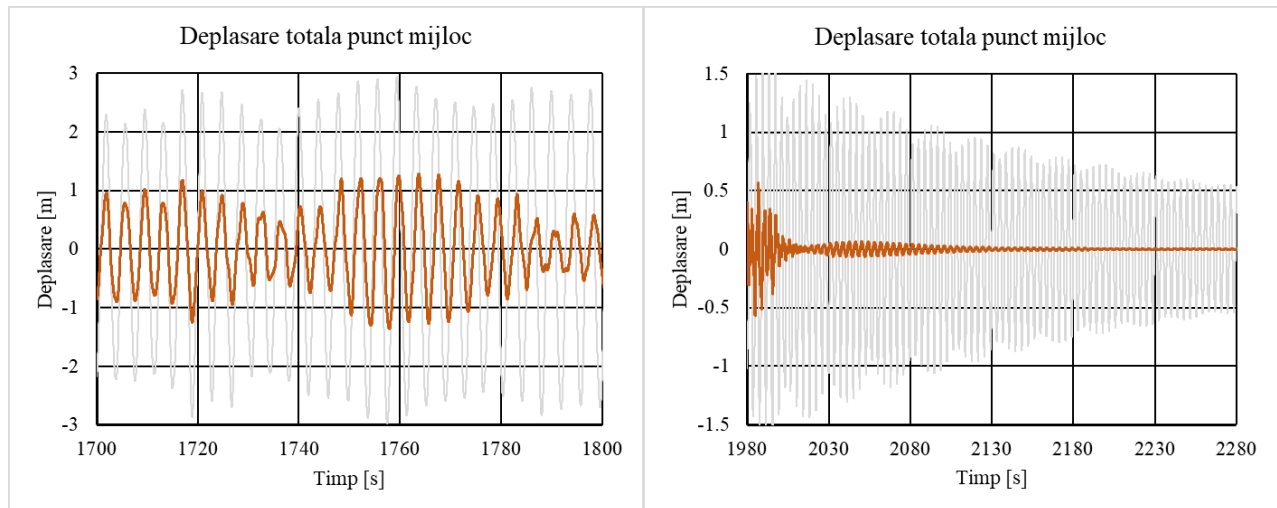


Figure IV-15 – Graphs with the displacement of the middle of the cable over smaller time intervals

By analyzing the displacements in the middle of the cable, one can observe reductions in amplitudes at about 35% compared to the base scenario. There is an interval at the beginning of the analysis of about 50s in which the device does not seem to be working at all. This is due to the fact that there has not yet been a sufficient phase shift between the 2 cables and so, there are no relative displacements between the two. The maximum amplitudes oscillate around 2.00m, and the maximum is 2.60m.

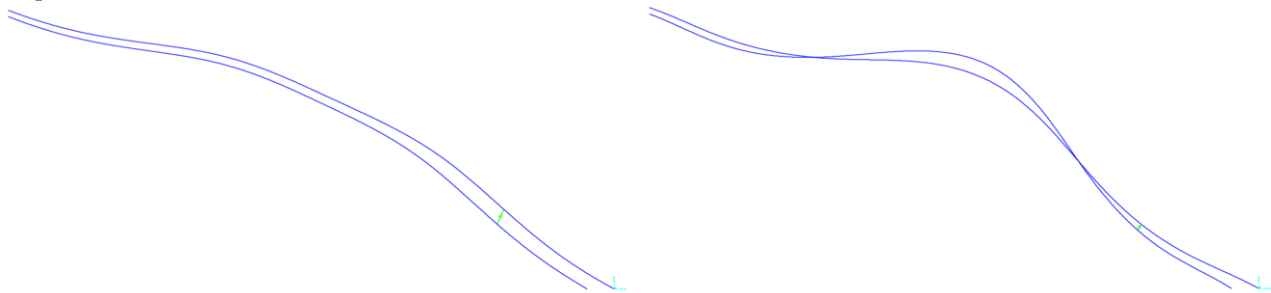


Figure IV-16 – Deformed shape of the cable stays corresponding to second 20, respectively second 1725 in the optimal case

Unlike the case of viscous dampers fixed to the superstructure that did not influence the frequency content of the oscillations and only decreased the amplitudes, the dynamic response of the cable in this scenario differs significantly. There is a phase shift that increases with the passage of time, reaching about five seconds towards the end of the analysis. The overlaps between frequencies are done differently, thus obtaining the peaks of movement at different times when compared to the other scenarios.

During the interval of free vibrations, the decrease of the oscillations to at an insignificant level of amplitude occurs much faster, over an interval of less than 20s.

In order to highlight the behavior of the device and its influence on the system, the hysteresis curves of the dampers positioned at 5%, 20% and 50% of the cable length are also presented below.

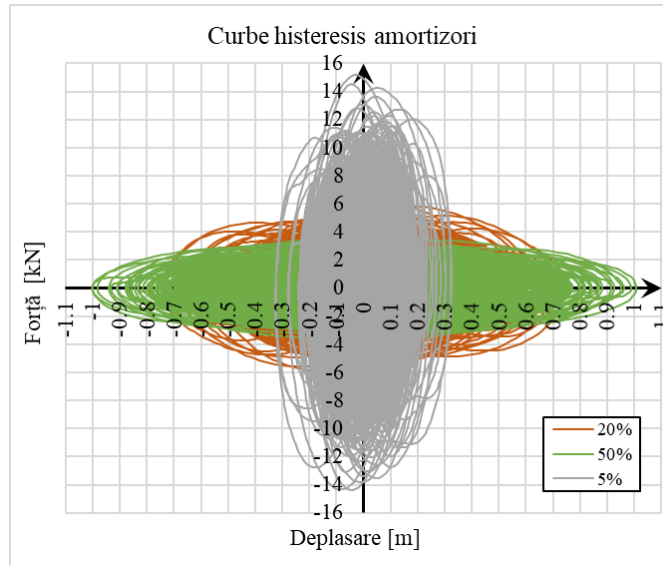


Figure IV-17 – Dampers hysteresis curves

Strokes of the order of $\pm 1.00\text{m}$ are observed for the device positioned in the middle of the cable, compared to the much lower values of only $\pm 0.25\text{m}$ corresponding to the case in which it the damper was positioned at 5% of the cable's length. This may prove to be a problem for the device itself. With regard to the forces generated by shock absorber in the cable, the situation is reversed, with values higher than $\pm 14\text{kN}$ in the case of the device located towards the end and only $\pm 3\text{kN}$ in the case of the device in the center.

However, the area described by the shock absorber hysteresis positioned at 20% of the cable is larger, suggesting a higher amount of dissipated energy and thus, a higher critical damping percentage.

Based on the above, a partial conclusion can be drawn regarding the solution for fixing the shock absorbers between the cables, which are more efficient for positions in the range of 10% -25% of the cable length. This will be considered for the shock absorber arrangement schemes in the next chapter.

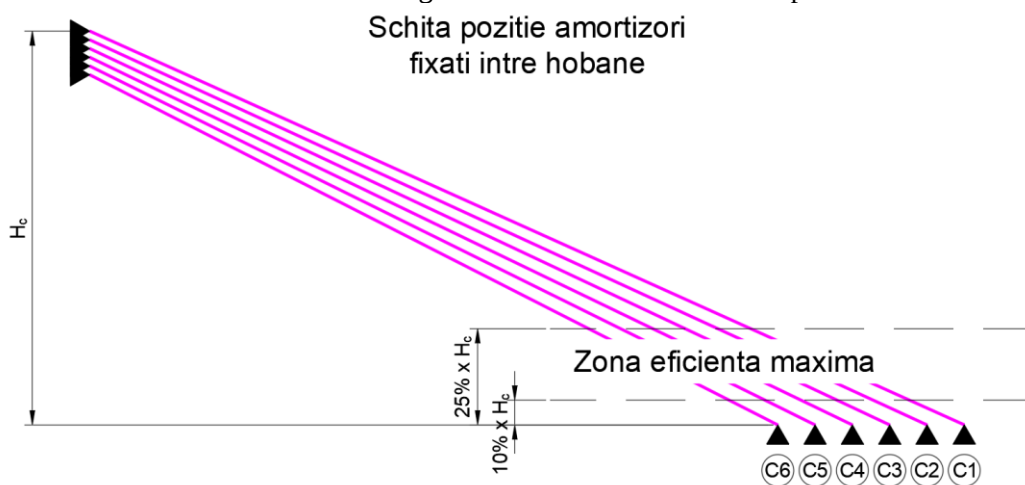


Figure IV-18 – Maximum efficiency zone for viscous dampers fixed between adjacent cables

II.5.2. The influence of relative positions of dampers fixed between adjacent cables

In this last stage of the case study, the group effect that occurs when there are several dampers and the influence of their arrangements on the dynamic response of the cables are analyzed. In this regard, 4 arrangements were considered for the 5 devices fixed between the 6 hobs. In order to determine the position of the devices, it was intended that they be within the maximum efficiency range determined according to the previous chapter. Thus, the following arrangement schemes were chosen:

- Layout scheme 1 - all 5 dampers were placed collinearly, in a direction perpendicular to the bisector between stays C1 and C6; their relative positions along the cables vary between 18-22% in length, so that the center of the segment formed by them is located at about 20% of the maximum height of the studied stays (H_c);
- Layout scheme 2 - the dampers were placed collinearly, in a horizontal direction; in view of the similarity with the previous scheme, they were placed at a height of 20% of the maximum height of the cables (H_c);

- Layout scheme 3 - the dampers were placed individually, each perpendicular to the bisector between the 2 stays to which it is fixed, with the centers of the devices at a height of about 20% of the maximum height of the cables (H_c);
- Layout scheme 4 - the dampers were placed similarly to scheme 3 from the point of view of the direction, but the arrangement was staggered, on 2 horizontal planes located of 15%, respectively 25% of the maximum height of the cables (H_c);

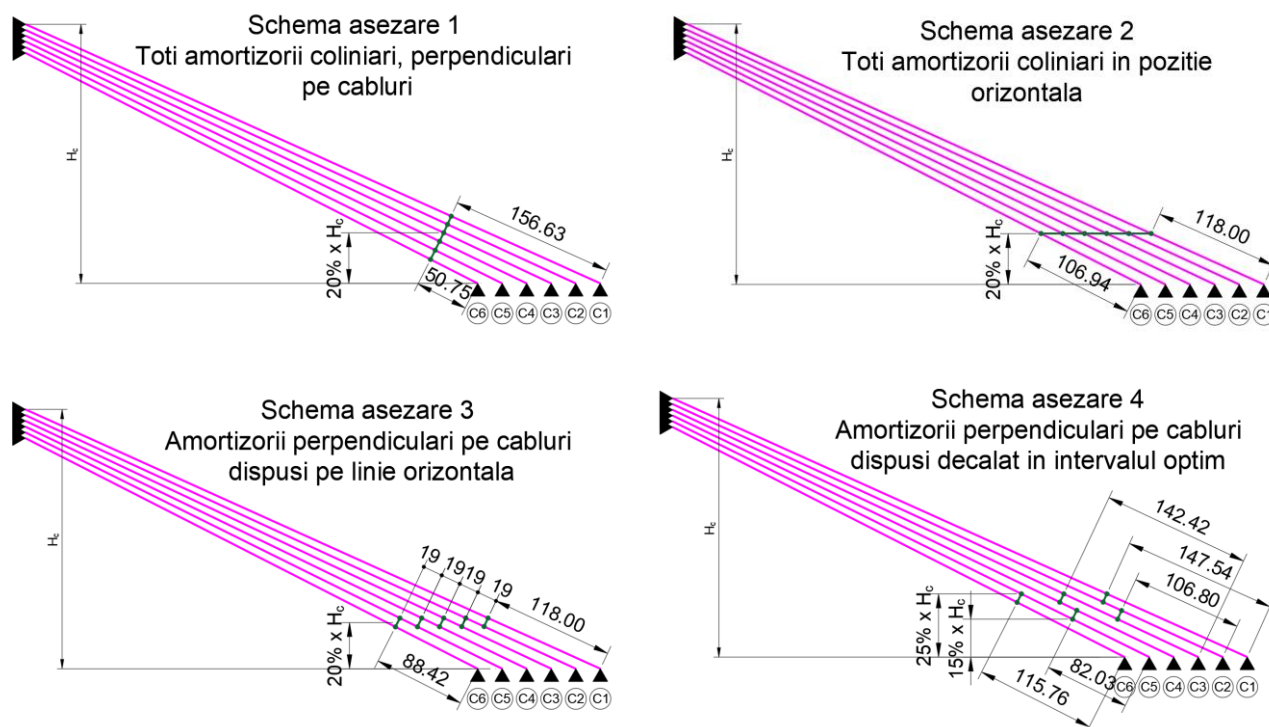


Figure IV-19 – Damper layout schemes considered

For each of the scenarios presented above, several analyses were performed in order to determine the value of the viscous coefficient for which the dynamic response of the cables group is optimal. The aim was to find the value of the optimum viscous coefficient corroborated with the optimal position of the devices. As a result, only the dynamic response of the C3 stay is presented, its behavior being similar to that of the other cables. It was necessary to model all 6 cables to simulate the group effect. For that cable, the maximum force generated by the damper between cables C3 and C2 was determined, the percentage of the critical damping resulting for the whole system and last but not least, the resulting Scruton number for cable stay C3. The following table shows the results of all these parameters, corresponding to the most efficient situations, as they emerged from the calculations.

Tabel 2 – Analysis results for the group of dampers

Poziție amortizor în lungul cablului	Vâscozitate optimă		Deplasar e maximă cablu	Forța maximă amortizo r	Procent din amortizare a critică ξ	Scruton
	Coeficient de vâscozitate c	Procent din c optim teoretic				
	[kN x s / m]	[%]				
Schema 1	50	133.00%	1.029	22	2.11%	15.640
Schema 2	50	133.00%	1.081	17	1.92%	14.240
Schema 3	50	133.00%	1.016	24	2.16%	16.013
Schema 4	50	133.00%	1.027	18	2.11%	15.699

Compared to the dynamic cable responses from the previous chapter, all the proposed arrangements showed more positive results, but the most efficient arrangement was the one proposed in Scheme 3. The results of this analysis are presented in detail in the following graphs:

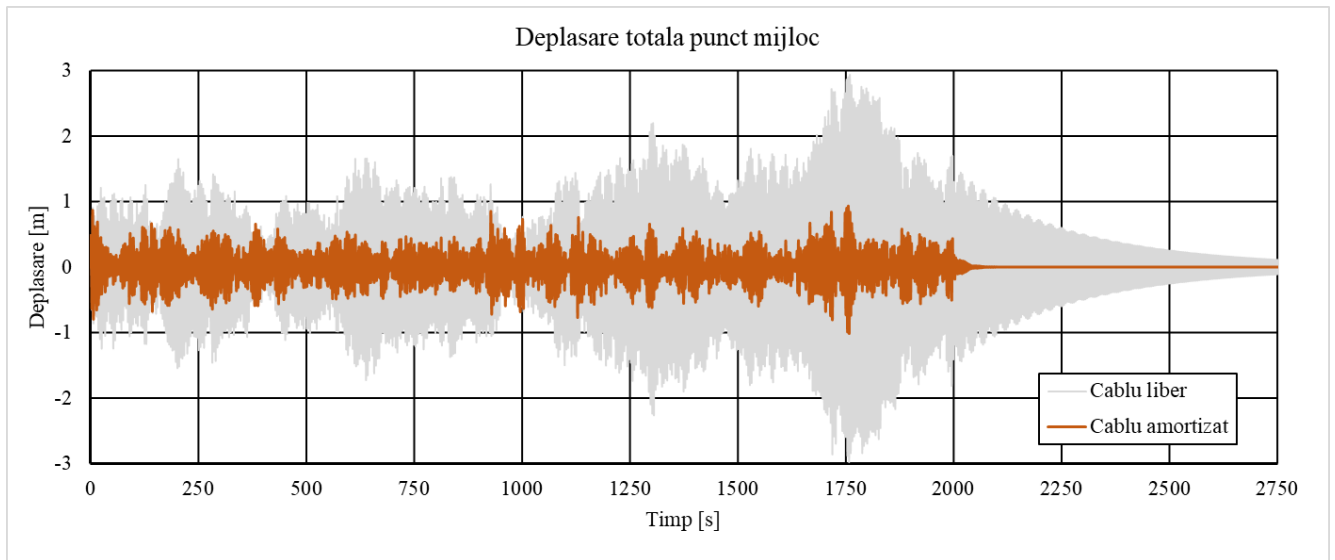


Figure IV-20 – Graph representing the displacement in time of the point in the middle of the C3 cable

The maximum displacement amplitudes of the center of the C3 cable are around 1.20m, with local peaks in the threshold of 2.00m. The absolute maximum was also obtained around 1750s and is 2,032m.

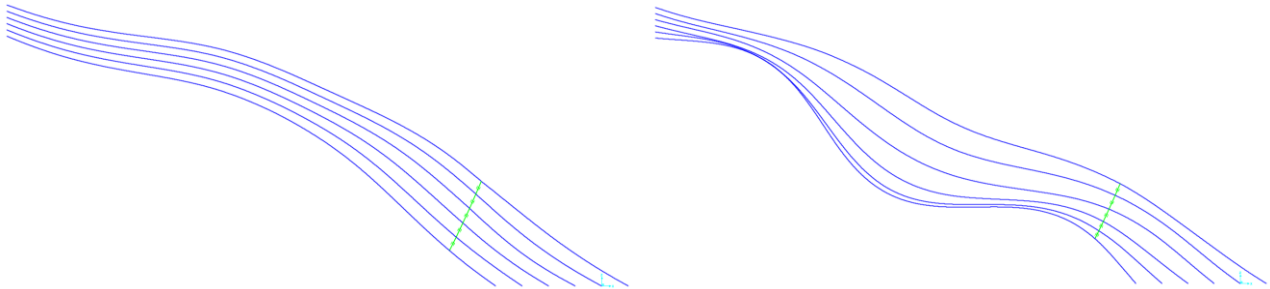


Figure IV-21 – Deformed shape of the cable stays at seconds 20 and 1725 in the optimal situation

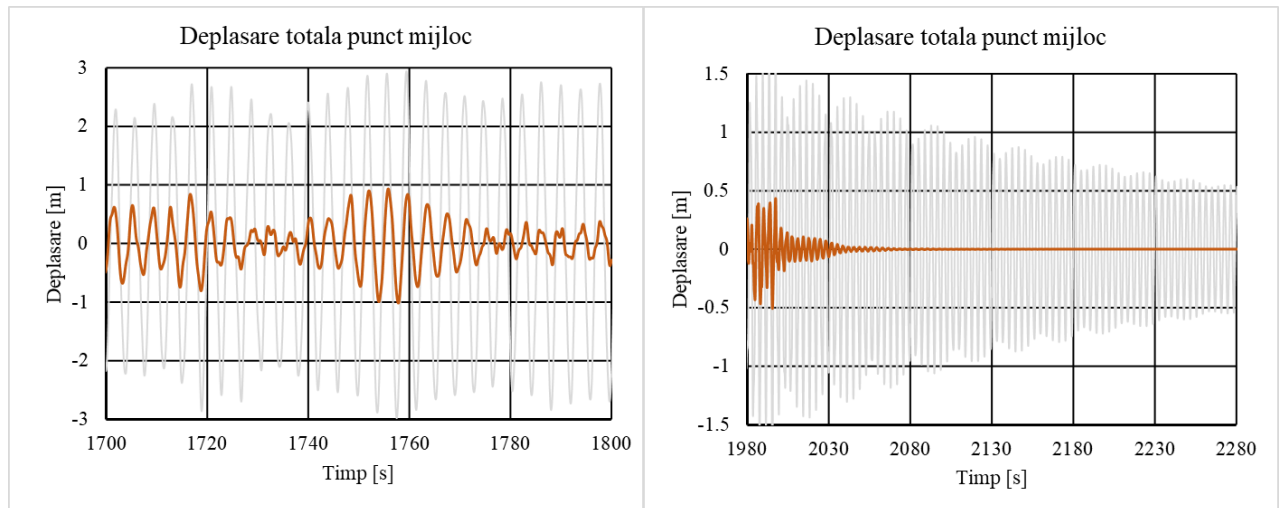


Figure IV-22 – Graph showing the displacement in the middle of the C3 cable over short intervals of time

After a sturdy analysis of the movement of the C3 cable it was found that when compared to the reference situation, a small phase shift between the two cases can be observed, which leads to a significant delay in the movement of the two and the appearance of an additional oscillation cycle at about every 100s. This would suggest a decrease in the vibration period and thus a significant increase in the stiffness of the system. Also, by comparing the speed of the movement, it can be seen that the dynamic response differs greatly from the initial one, having other shapes of the macro cycles, although the maximum amplitudes are reached at approximately the same times.

After stopping the external excitation, it takes less than 30s for the amplitudes to drop below the threshold after which they become insignificant. It is important to note that the very low level of amplitudes after only 30s is not reached in the version without dampers even after the 750s until the analysis stops.

The behavior and operating parameters of the 5 modeled viscous dampers are similar, which is why only one will be presented below. In order to better show how the devices work, the hysteresis curve of the device fixed between stays C3 and C2 is shown below:

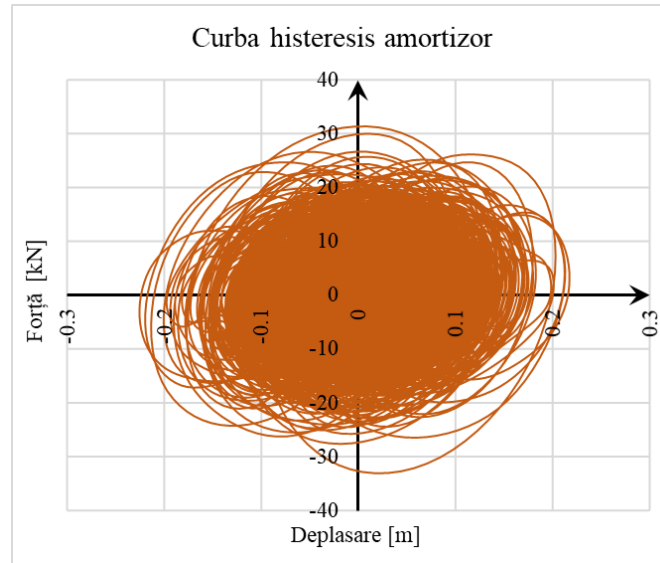


Figure IV-23 – Hysteretic curve of the damper between cables C2 and C3

As in the other 2 cases, the hysteresis curve is ovoid, with force values around $\pm 30\text{kN}$ and displacements of up to $\pm 20\text{cm}$. Compared to the other 2 cases analyzed, the force values are similar to those for the damper fixed to the superstructure, but much higher than those of the individual dampers fixed between only two cables. In a similar manner, the displacements are higher than in the case of those fixed by the superstructure, but much smaller than in the case of the individual ones between the cables.

In all the arrangement schemes studied, values of the Scruton number of over 10 were obtained, being considered acceptable according to the imposed criteria. The percentage of critical damping in cables reaches values of about 2%, which can be considered very good considering the characteristics of the studied structure.

CHAPTER III FINAL CONCLUSIONS, PERSONAL CONTRIBUTIONS AND FUTURE STUDY DIRECTIONS

In modern engineering practice, most solutions for cable stays vibration mitigation involve the attachment of external dampers to the superstructure, fixed between it and cables, in order to dissipate the energy given by the oscillations of the cables. One of the main drawbacks of this solution is the position of the devices, which is very close to the ends of the stays, in a zone where their efficiency is low. For the best results, they should be fixed between the superstructure and the middle of the cable, but this cannot be done for physical reasons due to the very large dimensions that would result for the devices.

Nowadays, there is a wide range of cable vibration damping systems worldwide, many of which have been used successfully in structures and have given acceptable results. The most common problems are related to the costs, which are quite high, of the mounting which is often very difficult due to the fact that they are usually placed in hard to reach positions and last but not least, the resistance over time for which, until now, there is not enough data to be able to draw any relevant conclusions, these devices being relatively new, at about 25 years old.

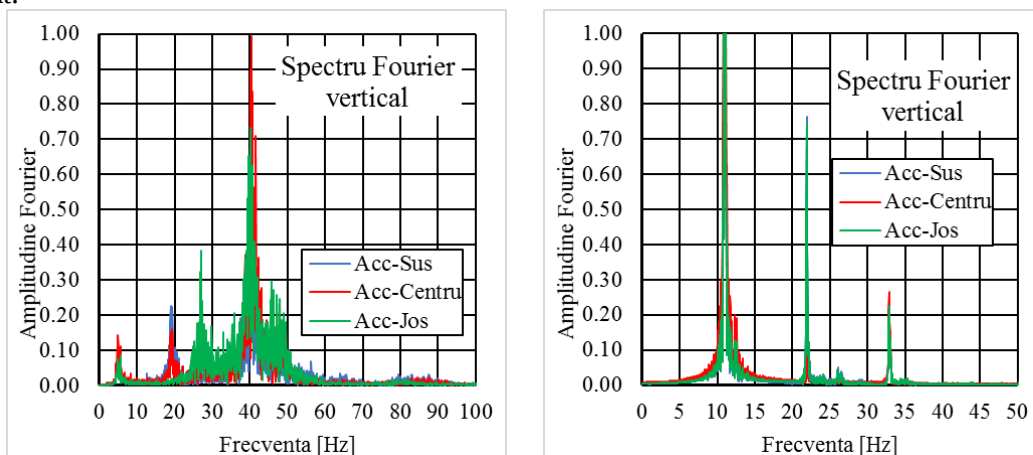
Even with today's high-performance computing tools available, estimating the dynamic response of cables with vibration mitigation devices is very difficult to do and requires a great deal of computing power. One of the objectives of this paper was to propose a simplified algorithm for determining the dynamic response of cables to external excitations caused by weather.

The approach proposed for solving this issue involves a series of simplifying hypotheses, including the breakdown of the phenomena in two separate branches of physics, fluid mechanics and structural mechanics. For the fluid mechanics part, two-dimensional calculation models with triangular and quadrilateral finite elements, specific to the field, were made, from which the external excitation that should be applied on the cable was extracted, as well as its variation in time. The best representation was in the form of an evenly distributed force over the entire length of the cable and sinusoidal variation in time, with periods depending on the diameter of the outer sheath. In order to determine the cable response in different situations, the principles of structural mechanics were considered, based on two-dimensional models with bar-type finite elements and specialized finite elements with hysteretic behavior in the case of dampers were made.

To validate the results, this approach was initially used to determine both the theoretical and the actual response of a small-scale cable that was instrumented and tested in the wind tunnel. After correcting all the parameters in the calculation models and applying the principles of similitude theory, the results obtained were quite close to the real ones, with errors below 5%. The proposed calculation method proved to be sufficiently accurate in estimating the dynamic response of the studied cable. There are of course differences in recordings, with values above and below those recorded, but generally with satisfying results.

The processing of the data from the measurements made on the cable in the tunnel resulted in a wide range of dynamic cable responses. There were random vibrations in the form of noise, stable vibrations in the form of oscillations with constant amplitude, resonance phenomena and beats phenomena. By processing the results, dominant frequencies of the movements were obtained, the influence of the lower and the upper modes could be seen and complex damping mechanisms were highlighted, all of which are difficult to model mathematically.

It has been observed that the dynamic response of the cables depends on the geometric and material characteristics, but also to a large extent on their internal tension. For lower tension levels, the input of the higher self-vibration modes has been found to be greater, while at higher axial stresses the fundamental mode becomes predominant.



Relevant frequencies at low tension Relevant frequencies at high tension
Figure V-1 – Fourier spectra with dominant frequencies at low tension levels, respectively at high tension levels

The internal damping of the cables also depends to a large extent on the internal state of stress and the level of the oscillations. At lower levels, based on the dynamic responses in the form of free vibrations, a fraction of the critical damping of 5.23% was obtained, while at higher levels it decreased to 1.11%.

Percentage of the yield limit	3.01‰	7.31‰	16.61‰
Maximum damping	7.16%	5.08%	2.40%
Minimum damping	3.30%	3.30%	0.79%
Average damping	5.23%	4.19%	1.11%

It was observed that under the action of the wind, the dynamic response of the cable is not always similar in the direction of air movement compared to that in the perpendicular plane to it. According to the records obtained, there were situations in which, in the direction of the wind (Z), the same cable vibrated in the form of two half-waves, and in the direction perpendicular to it (Y) vibrated in the form of three half-waves. For this reason, it is recommended to do a breakdown of the effects in two directions, one in the direction of air movement and one in a perpendicular plane to it when in estimating the dynamic response of the cable.

It has been found, especially at lower cable tension, that the frequencies of oscillations are dictated by the speed and direction of the air, and that they can overlap in different ways with the frequencies of the cable's own vibration mode. It is recommended that in estimating the dynamic response of the wind to the cables of the cable stayed bridges the worst situation of wind direction be considered. In the study, the direction was considered perpendicular to the plane formed by the cables with their horizontal projection.

The measurements also highlighted the effect of the anchorages on the dynamic response of the cables. According to the records, the accelerations from the bottom end of the cable were generally smaller than those from the top, a phenomenon visible especially during the increase or decrease of the air speed. This was most likely due to the active anchorage at the bottom, which was much more flexible than the passive one at the top, and influenced the vibrations in that area. For a more accurate estimation of the dynamic response of cable stayed bridge cables, both the local effect of the anchorage types and the overall effect of the cable-superstructure interaction, respectively cable-pylon, must be taken into account, as they can significantly alter the dynamic response of cable.

In the proposed approach, only the action of the wind was considered, but other phenomena that can induce excitations in cables must not be disconsidered, such as galloping, ice deposition, combination of wind and rain, traffic actions, etc .

Numerical analyses in both areas of physics (fluid mechanics and structural mechanics) have proven to be very sensitive to the integration step. In the case of the model in the field of fluid mechanics, the step was chosen according to the diameter of the stay and the wind speed, so that a theoretical particle crosses through the area adjacent to the obstacle in at least 20 steps, as recommended in the literature (Chanson H ., 2009). In the case of the model in the field of structural mechanics, the time step was chosen according to the significant periods of oscillation. In the model, the time step was chosen by dividing the period corresponding to mode 3 of vibration into 20 intervals. An attempt was made to obtain the best possible ratio between the required computing power and the accuracy of the results, provided that the latter was sufficiently high.

Damping also plays a very important role in getting the accurate results. This was taken into account by the Rayleigh method by determining the coefficients α and β by wind tunnel test, so that the calculated results are as close as possible to those measured. The best results were obtained by calculating these coefficients with the traditional formulas, considering a percentage of the critical damping of 0.5% for the frequencies of modes 1 and 2 of vibration.

It was observed that the errors obtained between the calculated and recorded values varied by reference intervals. At free vibration intervals they reached up to 20%, the mathematical model showing a higher damping level at low amplitude oscillations, but were below 5% in the maximum resonance intervals at stable oscillations, with calculated values higher than those actually recorded.

In view of the above, it can be stated that the proposed method of separate analyses in the two fields of physics, in this case fluid mechanics and structural mechanics, has generated satisfactory results and can be applied to estimate cable responses on a real scale. Also, in order to determine the dynamic response of the cables, both direct integration and the modal overlap method can be used. Simple bar elements can be used to model the cable, provided that a suitable state of internal tension is simulated in advance.

The case study was conducted on the Vladivostok Bridge, a structure in which both viscous and magneto-rheological dampers were adopted on the longest cables to control the level of vibrations in real time. For very long stays of over 500m, such as those of the Russky Bridge, the solution to mitigate vibrations by means of

viscous dampers fixed to the superstructure is no longer sufficient. More efficient alternatives are needed, such as the ones that are adopted by using semi-active dampers.

Thus, the study had two main objectives. The first was to simulate the dynamic behavior of the cables in the scenario in which there are no vibration damping devices, and the second to show the influence of the dampers on the response of the cables. Three solutions for vibration mitigation by means of viscous dampers were studied, as follows:

- set between the stays and the deck (conventional solution)
- set between adjacent stays on pairs of two
- set between adjacent stays on pairs of five

Parametric analyses were performed for all these situations, considering the variation of the viscous coefficient of the dampers, variation of their position, respectively of their arrangement between the cables.

The results of the study reveal the importance of introducing damping systems on the dynamic response of cable stayed bridge cables. At the main span of the 1104m study bridge, cables lengths are up to 600m, which have been shown to be very susceptible to wind-induced vibrations. In the absence of these systems, the free cables have a very low internal damping of about 0.50-1.00% which, in combination with the negligible bending stiffness, leads in certain situations to oscillations with very high amplitudes of displacements over 6.00m, respectively of the tension with a variation of more than 1000kN (approximately 15% of the idle value without traffic), resulting in dynamic amplification factors of more than 15.

The introduction of any of the studied damping systems led to significant changes in the dynamic responses of the cables, with substantial reductions in maximum amplitudes in terms of displacement or internal stresses, but also reductions in attenuation times of oscillations under free vibration.

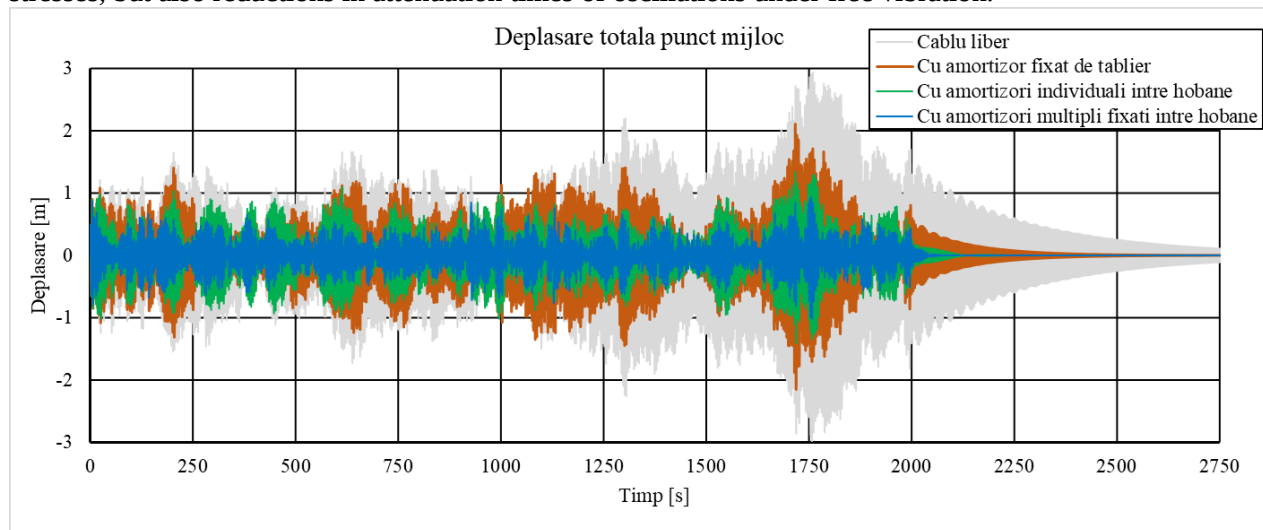


Figure V-2 –Comparison of the movement of the point in the middle of the cable over time

After comparing the results, it was concluded that the most efficient system was the one based on the installation of a group of dampers between adjacent cables. The maximum amplitudes in terms of displacement were reduced to 1.50m, which is less than 25% compared to the base situation, and the oscillations under free vibrations lasted about 30s, a very small fraction compared to the time interval of over 750s resulting on free cables. By comparison, the most commonly used solution of installing dampers at the base of the cables fixed to the deck, reduced the maximum amplitudes to about 4.00m, that is only 66% compared to the reference case, and the free vibration damping time was about 300s, 10 times higher than in the optimal solution.

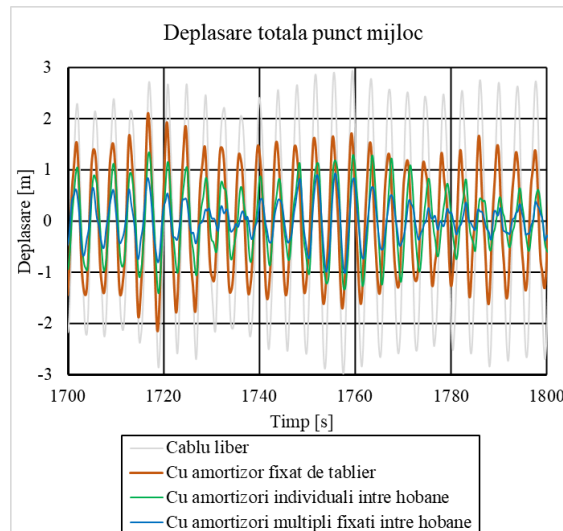


Figure V-3 –Short-term comparison of the movement of the point in the middle of the cable

By comparing the movements at local level at short intervals, it was found that the more favorable response is maintained in the case of dampers fixed between stays, either alone or in groups, compared to that of devices fixed to the deck. The movement of the point in the middle of the cable had similar paths in all four cases, with increases and decreases in amplitudes over the same intervals, but with differences given only in terms of the values of amplitudes.

Comparing the hysteresis curves of the dampers in the 3 cases, we can see larger areas described by the 2 scenarios in which the devices are fixed between the cables, which indicates a higher amount of dissipated energy.

However, it must also be taken into account that the strokes of the dampers mounted in pairs of two are of the order of ± 65 cm, compared to only ± 20 cm in the case of the group of dampers and ± 10 cm in the case of those fixed to the deck. Theoretically, this is not a problem, but the physical realization of a viscous damper with a piston stroke of 1.50 m and very low viscosity could prove to be extremely difficult, with very high costs that could make the solution unfeasible. The behavior of the devices in the other 2 situations leads to parameters that are much easier to reach in reality, which can lead to much higher efficiency over price ratios.

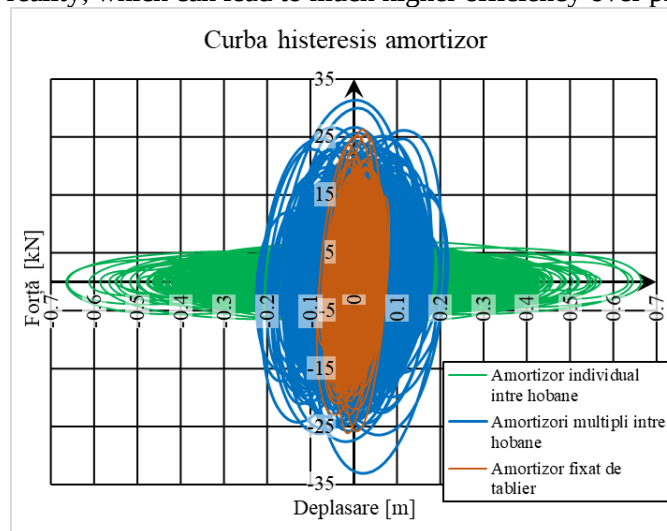


Figure V-4 –Damping hysteresis curves comparison

In order to obtain the best possible dynamic response, the viscosity of the damper must be chosen in accordance with the mass of the stay, the length, the internal voltage, the actual vibration mode targeted and the position of the device along the cable.

In the case of the dampers fixed to the superstructure, the efficiency rises with the increase of the distance of the cable fixing point along the stay, and the actual modes considered are between 1 and 5. Depending on these, specific literature (Wang X. Y., 2004) prescribes relationships for obtaining the optimum viscous coefficient.

In the case of devices fixed between stays, the approach is not as simple. Damping here also depends on the periods of oscillations that strongly influence the efficiency of the mitigation devices. For example, their efficiency is greatly reduced if they are positioned near the inflection points of the deformed cables. The cables oscillate in the form of half-waves and therefore these points can be determined according to the periods of

external excitations that overlap with those of their own periods of vibration. Considering this phenomena, an analysis of vibrations in the frequency range is useful, which is why the paper presents Fourier spectra and power spectra for vibrations in the 4 cases analyzed.

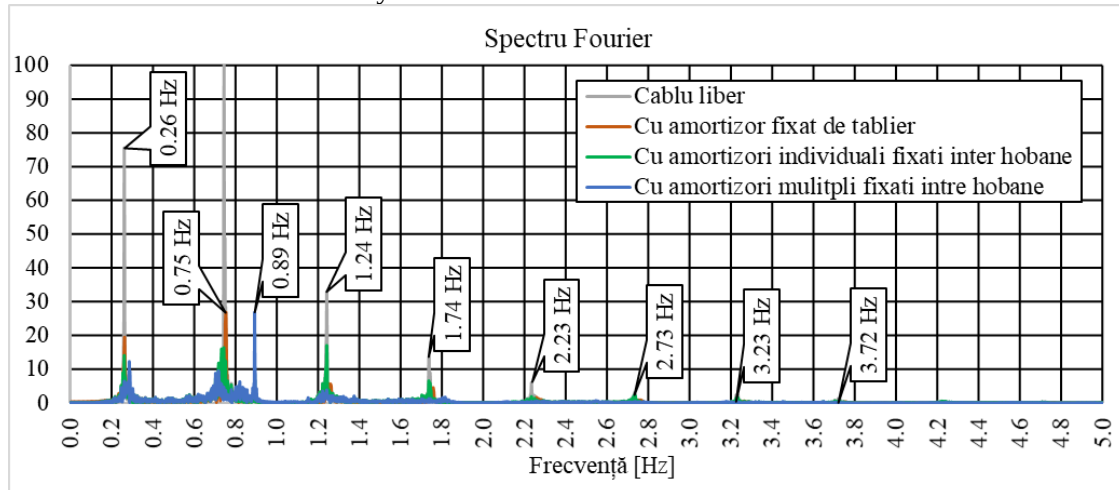


Figure V-5 –Fourier spectrum comparison

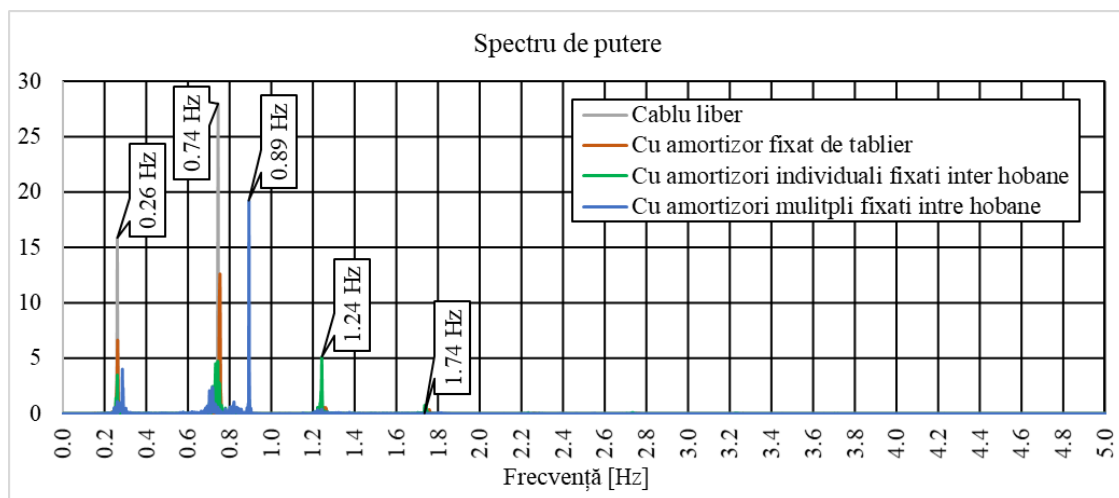


Figure V-6 –Power spectrum comparison

By studying the Fourier Spectrum, we can see the absence of vibration mode 2, with a frequency around 0.50Hz. The dominant frequency is that of mode 3, followed by mode 1, and then by the higher modes up to 9. In the case of the power spectrum, the same absence of mode 2 is observed, with fundamental mode being 3, but significant frequencies only to be found up to mode 5. This lack of vibration mode 2 is most likely due to the considered external excitation's time variation, which failed to induce vibrations at its appropriate frequency and does not necessarily mean that these cables will never vibrate by mode 2. However, it can be stated that there are external excitations that can induce vibrations in the cables only on certain frequencies and, most importantly, not always by the fundamental one.

In the scenarios with dampers fixed to the superstructure or with only one considered between adjacent stays, no changes of dynamic characteristics were found compared to the reference case, which was without devices. However, in the scenario of the group of dampers between the cables there is a shift in the frequency of mode 3 from 0.74Hz to 0.89Hz, most likely due to the interaction between the cables and the increase in stiffness under transverse displacements.

While searching for the optimal relative position along the cables of the dampers between pairs of two stays, it was found that the results were satisfactory for all situations between 5% and 50% of the length, except for the one at 33%, in which the influence of the device was insignificant and produced no substantial effects. This is most likely due to vibrations following the dominant mode 3, which has 3 half waves and 2 inflection points, one of which coincides with the position of the device.

Although the graphs with the variation of the amplitudes of oscillations in terms of displacements or axial forces over time in the cables can give a lot of information about their dynamic response, the most relevant efficiency comparison parameters are those based on the percentage of critical damping and the Scruton number, for which the literature recommends values above 10. (Sena Kumarasena N. P., 2007)

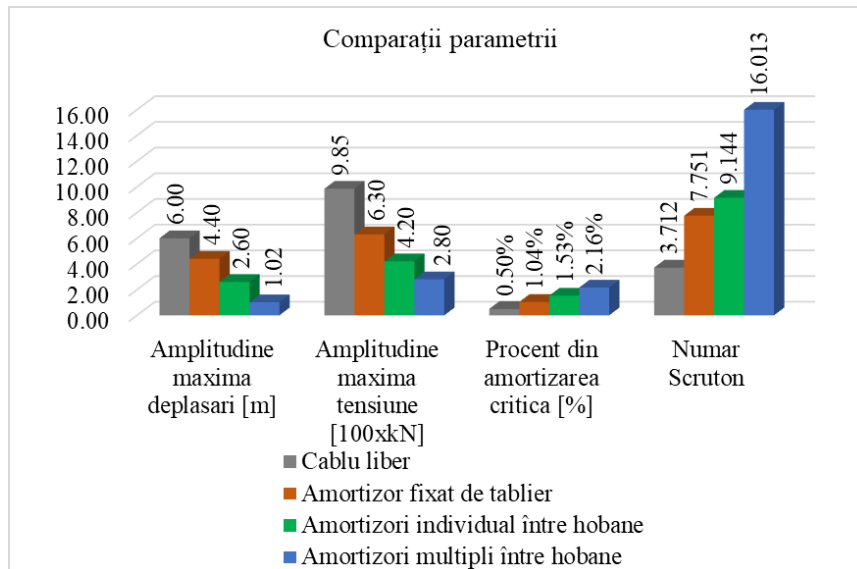


Figure V-7 – Comparisons on various criteria between cases

The introduction of external passive damping systems has led to a 100% increase of the percentage of critical damping in the case of devices fixed to the deck, increasing the value from 0.50% to 1.04%, respectively to an increase of 4 times the base value, to about 2.16%, in the case of those fixed between stays.

In terms of the values obtained for the Scruton number, the vibration damping devices led to significant increases compared to the base case, proportional to the damping. In the scenarios with dampers fixed to the superstructure, a value of only 7.75 was obtained, and in that of the devices fixed between the cables, values of 9.15 were obtained for isolated dampers, respectively 16,013 for those who work in groups. Compared to the prescriptions in the literature, only the last case would satisfy all the criteria demanded. This deficiency of the devices fixed by the superstructure is also confirmed by the solution actually adopted for the bridge. For the structure on which the study is based, passive viscous dampers were adopted only on the shorter stays, and for the longer ones semi-active magneto-rheological dampers were used, which are more efficient than the passive ones.

Although the best results from the study were obtained for the cases of the group of dampers fixed between the cables, it must also be taken into account that this scenario involves the highest costs both in terms of procurement and installation of devices and also in terms of maintenance of the structure over its lifetime.

It can be concluded that the solution based on viscous dampers fixed between stays and superstructure substantially improves their dynamic response, but for lengths of 600m as those in the study, this solution is no longer sufficient. In situations where this length is exceeded, it is advisable to adopt more effective solutions. One of these can be the one studied in this paper the proposes the installation of passive dampers between adjacent cables. This showed, at least in theory, very good results. Also, the 2 vibration mitigation methods presented are not mutually exclusive, they can be installed in parallel to obtain even more favorable dynamic responses. The choice of a damping solution must be made on the basis of stability criteria for aerodynamic phenomena, but also on economic criteria, so as not to lead to excessively high costs.

The case study showed that the solution of installing a group of dampers between adjacent cables can generate favorable results, provided that they are tuned and placed correctly. The best results were obtained when they were mounted at a relative position between 10% -25% of the length of the stays, avoiding the inflection points corresponding to their first 3 own modes of vibration. Out of the four arrangement schemes studied, all showed good results, but the most efficient scenario was the one in which the devices were mounted almost perpendicular to the cables at a relative position corresponding to 20% of the cable length. By comparison to the same structure with the conventional solution of placing the viscous dampers at the base of the stays, the damping of the cables and the related Scruton number doubled, showing that it is a viable solution with very promising results.

Provided that their positioning along the cables is within the above range, the arrangement of the dampers is quite flexible, showing acceptable results in all the schemes studied. This is important because it opens up the possibility of arranging the devices on aesthetic criteria as well.

However, the study does not take into account the economic factor. Manufacturing and installation, but also the maintenance of devices is very difficult to do, which is why this option is only recommended for very large spans of bridges over 800m, where conventional solutions no longer give the same efficiency.

The results of the study are not sufficient to determine a simplified formula for calculating the optimum viscous coefficient or the most efficient arrangement of the devices between the stays. These were found through

parametric variations and many cases of finite element analysis. Several more similar studies could lead to more conclusions regarding their behavior and possibly to finding some simpler relationships for manual calculation.

Only a single external excitation was considered in the study, for all analyses, in the form of a force evenly distributed perpendicular to the cable, along its entire length. Given the variability of the wind action in the three directions in space, an essential role should be provided by the coupling between the action in the horizontal direction and the variation in height of the intensity, aspects that were not taken into account. The efficiency of the devices may decrease or even increase for other forms and variations over time of the forces acting on the cables, depending on the frequency content of the vibrations. Also, only plane analyses were performed, neglecting the movements in the direction perpendicular to the plane of the stays. Several studies should be performed to determine the influence of vibrations on the other direction on the cable response as well.

The study only deals with the option using viscous dampers between cables, but other hysteretic devices such as those with elastomers, friction or even magneto-rheological ones can also be considered. It is also possible to consider the possibility of combining the 2 solutions, the conventional one with dampers fixed to the deck and that of the devices between stays, with potentially even better results.

These aspects may constitute future directions of research on a larger number of bridges with similar or even different structures, by reference to the structure studied in the paper.

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