DAMAGE EVALUATION AND CONTROL FOR FATIGUE UNDER STOCHASTIC EXCITATION

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Abstract

In the paper we consider the problem of fatigue damage arising on light alloy space frame towers subjected to wind induced vibrations during the service lifetime. In order to mitigate the fatigue damage, a passive control technique is proposed, based on the implementation of viscoelastic dampers.
A full stochastic formulation is adopted for wind loading, vibration calculation and fatigue analysis, taking advantage of Eurocode 1 and 9 technical supports; the response calculation is performed in the frequency domain under the hypothesis of linearized drag and lift force spectra, and linear elastic structural behavior. Among the many methods available in the literature for fatigue analysis, the Rayleigh method is selected for calculations. However, some comparisons made with the Rainflow algorithm show that in order to achieve an unbiased life evaluation, the intrinsic error of the Rayleigh approximation must be accounted for with a calibration. The obtained results show that aluminum lattice towers may be very sensitive to fatigue problem. In the paper a design procedure is outlined that allows to estimate the expected fatigue life of the structure and to assess the optimal damping ratio required in order to extend the structure service life up to the target one.

Introduction

This study considers the problem of fatigue damage arising on an aluminum space frame tower subjected to wind induced vibrations during the service lifetime. In order to mitigate the fatigue damage, a passive control technique is proposed based on the implementation of a number of mechanical devices characterized by viscoelastic behavior (Aprile, Inaudi and Kelly, 1997).
A full stochastic formulation is adopted for wind loading, vibration calculation and fatigue analysis, taking advantage of Eurocode 1 and 9 technical supports. Among the many methods for fatigue analysis available in the literature, the Rayleigh method and the Rainflow rule are selected for calculations (Lutes and Sarkani, 1997). As is well known, the Rayleigh method works out a damage index affected by an unpredictable error, in case of non narrow-band spectra (Khil, Sarkani and Beach, 1995). However, the Rainflow analysis carried out on a set of simulated histories can be used as a calibration tool in order to refine the Rayleigh approximation.
The obtained results show that the aluminum lattice tower under study is quite sensitive to fatigue problem, although its safety at ultimate limit state conditions is completely fulfilled. So, it is necessary to quantify the damping required to extend the estimated fatigue life up to the target value. With this aim, in the paper the authors present an approximate design approach capable to identify the required equivalent damping starting from a set of structural element curves relating the damage indices to the service lives.

Aluminum lattice tower structure

The aluminum lattice tower considered in this study is shown in fig. 1. The tower is 24 m tall and is equilateral triangular in plan with sides of 6 m in width. The reticular scheme is modular: it is composed by 13 horizontal equilateral triangular frames, connected by 10 struts for each story; each tower story is braced by three diagonals in each corner triangle and three stabilizing space diagonals inside the core around the symmetry axis. The upper half height of the tower carries advertising boards, which cover approximately half of the structural free surface.

Figure 1. Lattice Tower (1999)

The structural system is patented as GEO system by CPM Sistemi, Florence, Italy. The structure is composed by tubular bars and spherical joints connected by means of a high strength steel bolt; the bars are worked out with AAA 6061 alloys extruded tubes in T6 status (conventional yielding stress f_{02} = 240 MPa) while the spheres with AAA 6082 alloy (f_{02} = 260 MPa). All the details of the system can be found elsewhere (Aprile and Benedetti, 1998).

The loading system consists on dead and live loads; the dead part includes the self weight of the frame and the added board's weight (1.0 KN/m^2) applied on the upper half part of the tower. The live load consists on the wind action applied on windward side nodes only, implicitly taking into account the wind action on leeward side.

Thanks to the reticular nature, the lattice tower bars are subjected uniquely to a uniform axial force distribution; so, the loading capacity of each single structural element must be evaluated deciding whether the stress is compressive or tensile. According to (Eurocode 9, Draft 1996) requirements, the loading capacity $N_{t,Rd}$ for bars in tension at ultimate limit state can be evaluated and verified as follows (Aprile and Benedetti 1998):

$$N_{t,Rd} = r_b A_b f_{02}, \quad \frac{N_{t,Rd}}{N_{Sd}} \geq g_M, \quad g_M = 1.1, \tag{1}$$

where A_b is the bar cross section and r_b=0.6 is a reduction factor due to the stress amplification at the bar end connection; N_{Sd} is the design axial tension. Analogously, for bars in compression the loading capacity $N_{c,Rd}$ can be verified as follows:

$$N_{c,Rd} = \bar{N}(l_b) A_b f_{02}, \quad \frac{N_{c,Rd}}{N_{Sd}} \geq g_M, \quad g_M = 1.1, \tag{2}$$

where $\bar{N}$ is a buckling coefficient, function of the bar slenderness.

Stochastic analysis of the structure under wind excitation

The wind velocity can be decomposed into a constant average velocity and a fluctuating velocity that can be properly described as a stationary random process varying along the height of the tower. For vibration analysis, the only fluctuating wind must be considered, and tanks to the structural symmetry, the wind turbulence in along-wind and cross-wind directions are not correlated (Simiu and Scanlan, 1996). The well-known Solari wind velocity spectrum is adopted for turbulence as a function of the frequency and the height (Solari, 1987); all the spectrum coefficients have to be derived by on-site meteorological conditions (Eurocode 1, 1994).

Once the loading system is known, the modal stochastic analysis is performed taking into account three significant vibration modes; thanks to stationary property of the process under consideration, the modal equations can be solved in the frequency domain. The total response is obtained in terms of power spectral density by superimposing the modal responses; the response cross-terms must be considered, due to the existence of not well-separated modal resonant peaks. For the analytical derivation of the displacement and stress averages and variances, as well as the expected value of the response largest peaks occurring in the gust time interval, reference is made to Simiu and Scanlan (1996). Finally, the problem of modal damping coefficient identification must be faced. The total modal damping coefficient is composed by three different contributions: structural damping, aerodynamic damping and damping added by means of dissipation devices. For lattice towers the structural damping coefficient is usually assumed around 1% as in Eurocode 1 (1994). The aerodynamic damping is originated by dissipative forces and it can result significant, in comparison to structural damping, for very flexible structures only. On the contrary, the addition of damping devices brings up a considerable modal damping increment whose computation can be carried out in terms of equivalent damping. The dynamic analysis results are reported in (Aprile and Benedetti, 1999).

Stochastic analysis of fatigue

The dynamic effects of wind may lead to material fatigue damage, which is cumulative in nature and non decreasing, so that the accumulation of damage continues until failure comes. Typically, the damage level is described by means of a damage index *D*, that is presumed to hold zero for a new structure and is normalized to unity when failure occurs. Due to the stochastic nature of wind loading, fatigue damage can be predicted in terms of damage probability, i.e. damage mean m_D and variance s_D^2, with s_D much smaller than m_D (Lutes and Sarkani, 1997).
In Aprile and Benedetti (1999) the damage statistic parameters for the examined structure are computed following the Rayleigh (RY) method taking into account multiple states of wind. As is well known, since the RY method is based on the assumption that the process is narrowband, the obtained results are sufficiently accurate only for this specific case; however, the method is widely applied introducing equivalent narrowband spectra suffering unpredictable error margins.
Following the Palmgren-Miner hypothesis of linear accumulation of damage, the following expectation of damage, can be evaluated for the whole design lifetime T_s:

$$E[D(T_s)] = (2pK)^{-1}(2)^{3m/2}\,\Gamma\left(1+\frac{m}{2}\right)E[T_s]\sum_r n_r\, s_{Sr}^{\,m-1}\, s_{\dot{S}r}\,, \tag{3}$$

where Γ is the Gamma function and $s_{Sr}, s_{\dot{S}r}$ are the standard deviations of stress amplitude and stress velocity respectively, evaluated for each acting *r*th state of wind. The definition of n_r is simply related to the *r*th loading process character, i.e. to the rate of occurrence of the *r*th state of wind. Typically, this last fits fairly well the Weibull distribution (Simiu and Scanlan, 1996). The factors *K* and *m* are the coefficients of the *S*-*N* curve, they depend on the structural detail material and geometry, and have to be determined experimentally. In the present analysis, K=13.44· 10^{18} MPa and m = 7 are assumed (Eurocode 9).
Finally, once the mean damage is evaluated through eq. (3), when the loading action does not depend on the design life, the expected fatigue life T_l can be easily derived:

$$E[T_l] = \frac{T_s}{E[D(T_s)]}. \tag{4}$$

Safety to fatigue is ensured when $E[D(T_s)]$ £ $1/g_f$, where g_f is a fatigue safety coefficient bigger than one, depending on code requirements. However, the wind loading is a function of the design life and so, in this case, the application of eq. (4) represents just a linear approximation of the actual non linear $T_l(D(T_s))$ function and can lead to meaningful deviations in the estimated fatigue life.

Rainflow analysis

In order to assess the error significance of the RY approximation for the considered wind analysis, the Rainflow (RF) algorithm is also run (Downing and Socie, 1982). A number of stress histories are simulated moving from the power spectral density (PSD) of each structural element; the adopted simulation technique is drawn from Soong and Grigoriu (1993) by adopting a sampling frequency of 0.01 Hz. In this case, a set of 10 stress histories lasting 3600 s is considered for the RF analysis, in order to evaluate the damage mean and standard deviation; it should be observed that the obtained results show a quite scattered distribution. However, this behavior is typical of both experimental and analytical fatigue assessments.

As shown in fig. 2, the generic element PSD exhibits a bimodal frequency content; the computed irregularity factor (IF) is in the range 0.8-0.9. The approximation inherent in the adopted simulation technique can be pointed out by superimposing the original PSDs with those obtained through the back analysis of the simulated stress histories. As can be observed in fig. 2, the diagram subtended areas are slightly different due to the numerical round off.

The comparison in terms of mean damage between the results of RY and RF analyses for the most critical structural elements is reported in fig. 3; remarkable differences between the obtained results are evident, being the RY damage from 1.5 to 3 times bigger than the RF damage. Therefore, despite the IF is quite close to unity (as is well known, for narrowband spectra IF = 1), the RY analysis strongly overvalues the expected damage. On the other side, it should be considered that the RF results are not totally error free and that, furthermore, the RF procedure is much more time requiring than the RY one. The actual problem of the stochastic fatigue analysis is the very poor confidence of the obtained results, no matter which method is used.

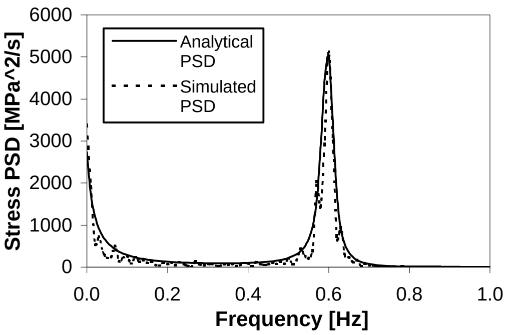

Figure 2. Analytical and simulated PSD

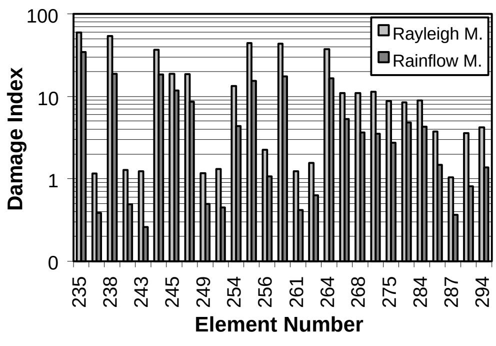

Figure 3. RY and RF damage indices

Effect of damping on wind induced fatigue life

It is well known that the addition of damping devices to structures subjected to dynamical vibrations can induce a remarkable suppression of structural vibration;

consequently, this vibration damping turns into a considerable reduction of structural fatigue damage. Among the several damping devices available on the market, the implementation of viscoleastic dampers results to be particularly simple and effective (Aprile, Inaudi and Kelly, 1997).

In a previous work (Aprile and Benedetti, 1999) the authors presented a detailed analysis of the effect of viscoelastic dampers introduction on the damage index of the structural elements composing the tower under study; in addition, the best damper characteristics and placements in order to minimize the fatigue damage were searched. By this way, the optimal damper system was set and the relative equivalent structural damping computed. It was observed that different damper layouts show a wide range of efficiency, but in any case there is a threshold of the added damping above which no further damage reduction occurs. Focusing on the best identified layout, which involves the introduction of dampers in the perimeter diagonals, it is possible to attain an equivalent structural damping around 10% with a reasonable viscoelastic material volume. From the optimal design point of view, it is relevant to assess the problem of selecting the minimum damping value necessary to upgrade the estimated fatigue life of the structure to the target design life.

Element	Estimated Life [years]			Damage Reduct. Factor [%]		Design Damping [%]	
	Linear (RYM)	NonLinear (RYM)	Modified (RFM)	NonLinear (RYM)	Modified (RFM)	NonLinear (RYM)	Modified (RFM)
235	0,8	6,3	11,0	12,6	22,0	4,4	2,5
238	0,9	6,6	18,8	13,2	37,7	4,2	1,4
244	1,4	7,8	15,5	15,6	31,1	3,6	1,7
245	2,6	10,8	17,3	21,6	34,6	3,5	1,5
248	2,7	10,9	23,4	21,8	46,8	3,5	1,0
254	3,7	12,3	37,9	24,7	75,8	3,2	0,4
255	1,1	7,2	20,7	14,3	41,4	3,9	1,2
258	1,1	7,2	18,1	14,5	36,2	3,9	1,5
264	1,3	7,7	17,5	15,4	35,0	3,6	1,5
265	4,6	13,6	28,1	27,2	56,2	1,9	0,8
268	4,6	13,6	40,6	27,2	81,2	1,9	0,3
274	4,4	13,3	42,9	26,6	85,8	2,0	0,3
275	5,7	15,2	48,6	30,5	97,1	1,8	0,1
278	5,9	15,5	27,4	31,1	54,7	1,7	0,8
284	5,6	15,1	31,2	30,1	62,4	1,8	0,7

Table I. Main data for the fatigue life estimation and damping ratio design

In order to obtain a correct estimation of the structural elements fatigue life, the nonlinear function relating the damage index to the service life must be constructed for all of them; for numerical efficiency reasons, this function can be built making use of the RY method, but a calibration is required using one or more RF calculations. In fig. 4 the damage index function is plotted for the critical structural elements; as is apparent, since the unitary damage index denotes fatigue failure, the estimated fatigue life is rather unsatisfactory in comparison to the target life, usually set to 50 years. The most critical structural elements for the considered lattice tower are the external vertical struts from the foundation level up to the middle height of the structure.

In table I the expected fatigue lives of critical elements computed by means of eq. (4) are reported in the first column while the expected lives obtained through the nonlinear functions (fig. 4) are listed in column two. Furthermore, in column three, the expected lives calibrated by means of the RF analysis performed at T_s = 50 years are collected. It is to mention that the difference between the linear, the nonlinear and the modified projections are significant.

In order to rise up the expected life to the target life, a damage reduction factor must be evaluated; in fact, when the damage index is relatively small (less than one), a linear approximation can be applied and the damage reduction can be obtained simply

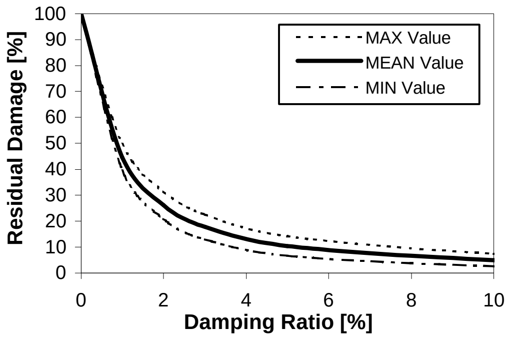

Figure 4. Damage of critical elements **Figure 5. Damage reduction with damping**

dividing the expected life by the target life. Then, a suitable damping ratio must be set in order to achieve this target damage reduction.

Making use of the RY method, the damage reduction for increasing damping ratio values can be easily computed. For the considered structural elements, the obtained results normalized to their maximum value are represented in fig. 5; as can be inferred from the figure, a single function can be derived for different elements, due to the small deviation of the data from their mean value. Entering now the diagram of fig. 5 with the requested damage reduction factors, the necessary damping ratio that has to be provided to the structure can be easily derived.

The obtained damage reduction factors and the relative design damping ratios are listed in table I. It is interesting to observe that the latter values are quite small, less than 5 %, whether computed by means of the RY or the RF methods; as a matter of fact, it is possible to provide the requested damping ratios with a moderate volume of viscoelastic material and with relatively small upgrading cost, matching finally the desired service life.

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