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Coupled Safety Assessment of Cable Stay Bridges

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Abstract

The life of the large stock of concrete bridges that still exist throughout the world must be extended, while at the same time ensuring that safety is not compromised for economic reason. In fact, environmental damage and fatigue life estimation of historical bridges represent a coupled structural issue for managing cost-effective decisions regarding rehabilitation or replacement of existing infrastructure, hence there is a need to estimate how long these structures could remain in service. In this paper, a coupled environmental and fatigue assessment of a representative cable stay bridge, is analyzed and assessed. Based on the analytical study, parametrical results on the lifespan of the bridge have been presented.

Keywords: bridge structure, concrete cable stay bridges, historical bridges, assessment

1. Introducation

The risk associated to the vulnerability of bridges and infrastructure is a relevant issue to guarantee standard safety and security of citizens in everyday life. ASCE Committee on Fatigue and Fracture Reliability (1982) reported that a relevant amount of bridge failures are related to fatigue and fracture, and this data is confirmed by Byers et al. (1997). The problem of fatigue assessment (Pipinato et al. 2011a; 2012b; 2012a) becomes further complicated if the deteriorated conditions of the existing structures need to be considered: defects of superstructures represent the 20.5% of causes for replacement of steel highway bridges (Nishikawa, 1997) while each year, about 1200 bridges reach the end of their design life (Yazdani & Albrecht, 1990). Most of them must be strengthened, repaired or rebuilt to ensure an acceptable level of safety considering present and future traffic conditions. Cracks originating from flaws could propagate under a time-varying random load process and the structural integrity is expected to degrade with time. When a fatigue crack grows up to a critical size, the structure could fail to be useful for its use (Zhao, 1996). These effects could be more significant when an exceptional event strikes the structure during its service life. In this context, according to authors' knowledge a correlation between fatigue effects and environmental actions is not deeply analysed in literature: so this is a crucial argument in bridge engineering, especially to estimate a precise cumulative effect of the total damage and to achieve the remaining life of historical infrastructures. Other existing approaches to the problem of lifetime performance prediction, for example, in the field of concrete structures affected by corrosion are shown in e.g in Biondini and Frangopol (2008; 2009). Moreover, interesting considerations on safety assessment under multi-hazards can be found in Duthinh and Simiu (2010). While the introduction on the matter could be found in Pipinato (2011), and in Pellegrino et al. (2011), a coupled approach, considering fatigue and seismic events is described in Pipinato et al. (2011b). Other studies on this relevant field have been presented in Pipinato et al. (2010) including the analysis and assessment of diverse existing steel bridges; in Pipinato and Modena (2010) a structural analysis of an existing arch metal bridge is reported; moreover, step level procedures have been implemented in order to give some insights on a rational procedure to be adopted in existing bridges (Pipinato, 2010); finally, experimental detail category have been studied and proposed by analizing the full scale testing results of dismantled bridges (Pipinato et al., 2008; 2009). Belonging to the aforementioned experience, in this paper a method for fatigue damage estimation of existing bridges in presence of environmental loading is presented, in particular dealing with cables in cable stay bridges. A companion paper analysis has been presented dealing with the design of new bridges, as illustrated in Pipinato et al. (2012). But this issue is relevant also in existing cable-stayed structures, suffering from the continuous aggression of environmental agents (urban, industrial, marine, etc.): these effects appear through corrosion, whose direct consequences are the strong modifications of the geometrical and mechanical characteristics of the components, and finally of the structure as a whole. A significant reduction of the bearing capacity of the cable with time, sometimes resulting in its partial rupture due to cyclic actions is often observed in these types of bridges. A large number of broken wires found in the suspension cables (Elachachi et al., 2006; Tanaka & Haraguchi, 1985; Xercavins & Mondorf, 1980; Elliott & Heymsfield, 2003) have shown the absence of methods for assessing safety levels provided by old suspensions. As a matter of fact, fatigue safety generally depends on the following three main parameters: the stress range due to traffic load (related to the structural behaviour of the bridge); the geometry of the construction details which leads to a more or less pronounced stress concentration and may trigger or accelerate fatigue crack propagation; the number of stress cycles due to the past traffic which directly influences the remaining fatigue life of the structure. The first part of this work deals with the description of the case study investigated; in the second part, a fatigue analysis adopting Eurocode provisions is outlined (EN 1993-1-9, 2005); finally in the third part, a damage coupled assessment including environmental damage is presented. A relevant assumption is that fatigue and environmental damage are not considered to interact in probabilistic terms, as for e.g. reported in other approaches (Duthinh & Simiu, 2010). The present study is focused on the damage estimation, and environmental induced damage coupled in deterministic terms. The approach can be considered as a first step towards a more sophisticated analysis of the problem which will take into account the interaction between fatigue and environmental damage as coupled processes in probabilistic terms.

2. Case Study

2.1 Bridge Description

The bridge across the Maracaibo lake in Venezuela is one of the largest in the world, with a total length of 8.678 m, and five 235 m cable stayed main spans (Figure 1).

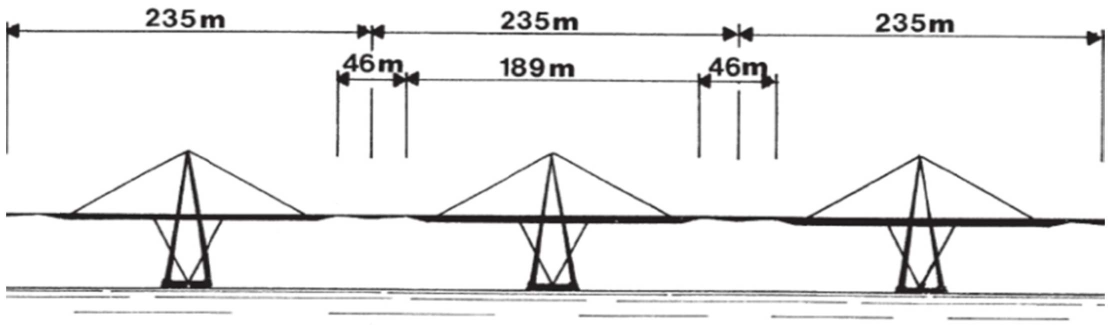

Figure 1. The Maracaibo Bridge: longitudinal view of the typical 235 main spans carried by a cable stayed structure, repeated for totally five spans across the entire bridge

This reinforced and prestressed concrete bridge structure ranks as one of the longest in the world (Podolny, 1973). The contractor was Maracaibo Bridge Joint Venture comprised of Precomprimido C. A., Caracas, Venezuela and Julius Berger A.G., Wiesbaden, Germany. The structural analysis and detailed plans were prepared by the Maracaibo Bridge Joint Venture in conjunction with the designer Prof. Riccardo Morandi. With the one exception, all the twelve designs submitted in the competition advocated a steel superstructure. The reasons for the selection of the Maracaibo Bridge Joint Venture design by the government commission were given as follows: reduction of maintenance costs as a result of the climate conditions existing at the site; the aesthetics of the design; the greater use of local materials and therefore less foreign exchange expended for imported materials; greater use of local engineering talent and labor. The five main navigation openings consist of prestressed concrete cantilevered cable-stayed structures with suspended spans having a total 771 ft (235 m) span (see Figure 2).

Figure 2. The Maracaibo bridge: central spans, cable stayed, main 235 m spans

Navigation requirements stipulated a horizontal clearance of 656.2 ft (200 m) and a vertical clearance of 147.6 ft (45 m). To preclude any possible damage as a result of unequal foundation settlement or earthquake forces, the central spans had to be statically determinate. Thus, a main span is divided into cantilever sections with a simply supported suspended center portion. The pier foundation width of 113.5 ft (34.6 m), Figure 3, was determined from transferring longitudinal bending moments, resulting from the placing of the suspended span and from unsymmetrical traffic loads.

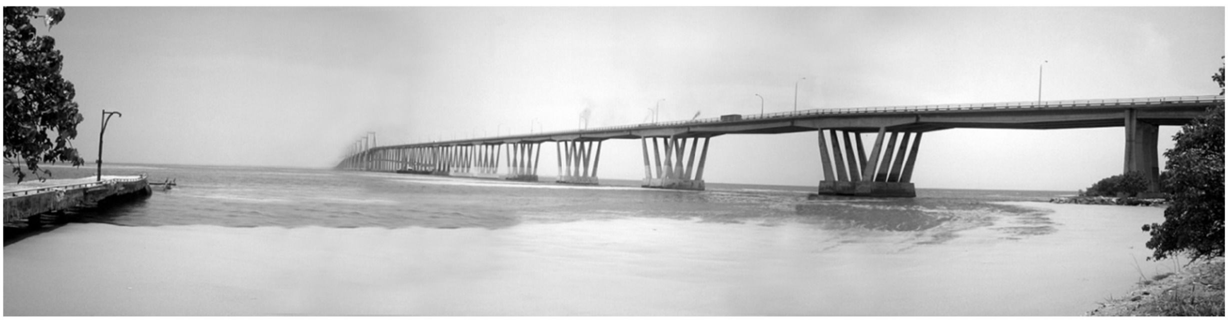

Figure 3. The Maracaibo bridge: approaching span

With a 656.2 ft (200 m) clear span requirement and a 150.9 ft (46 m) suspended span, the cantilever arm extends 252.6 ft (77 m) beyond the pier foundation. To avoid the large depth associated with a cantilever of this dimension, cable stays were used as a supporting system from the 303.5 ft (92.5 m) high towers. Any other system of support would have required deeper girders and thus would have changed the roadway elevation or infringed on the required navigational clearance. The cantilever span is supported on X frames while the cable stays are supported on two A frames with a portal member at the top. There is no connection anywhere between the X and A frames (see Figure 3). The continuous cantilever girder is a three-cell box-girder 16.4 ft (5 m) deep by 46.7 ft (14.22 m) wide. An axial prestress force is induced into the girder as a result of the horizontal component of cable force. Thus, for the most part, only conventional reinforcement is required. Additional prestressing tendons were required for negative moment above the X frame support and the transverse cable-stay anchorage beams. The pier cap consists of the three cell box girder with the X frames continued up into the girder to act as transverse diaphragms (see Figures 4 and 5).

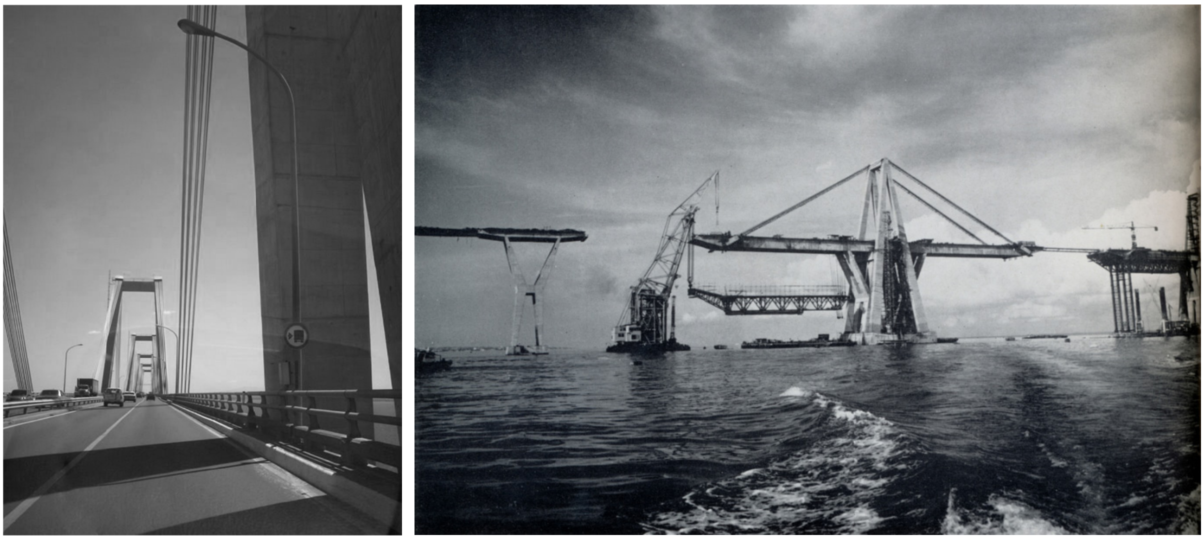

Figure 4. Maracaibo bridge, deck and pylon view, in exercise (left side); lifting construction deck operations (right side)

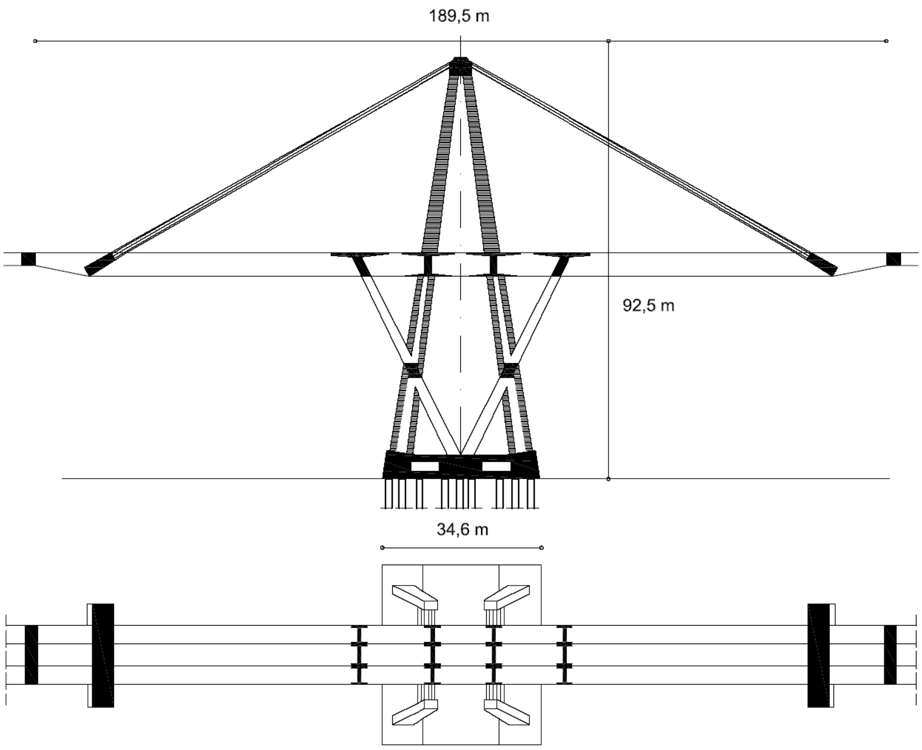

Figure 5. Main span tower and X frames, as built structure

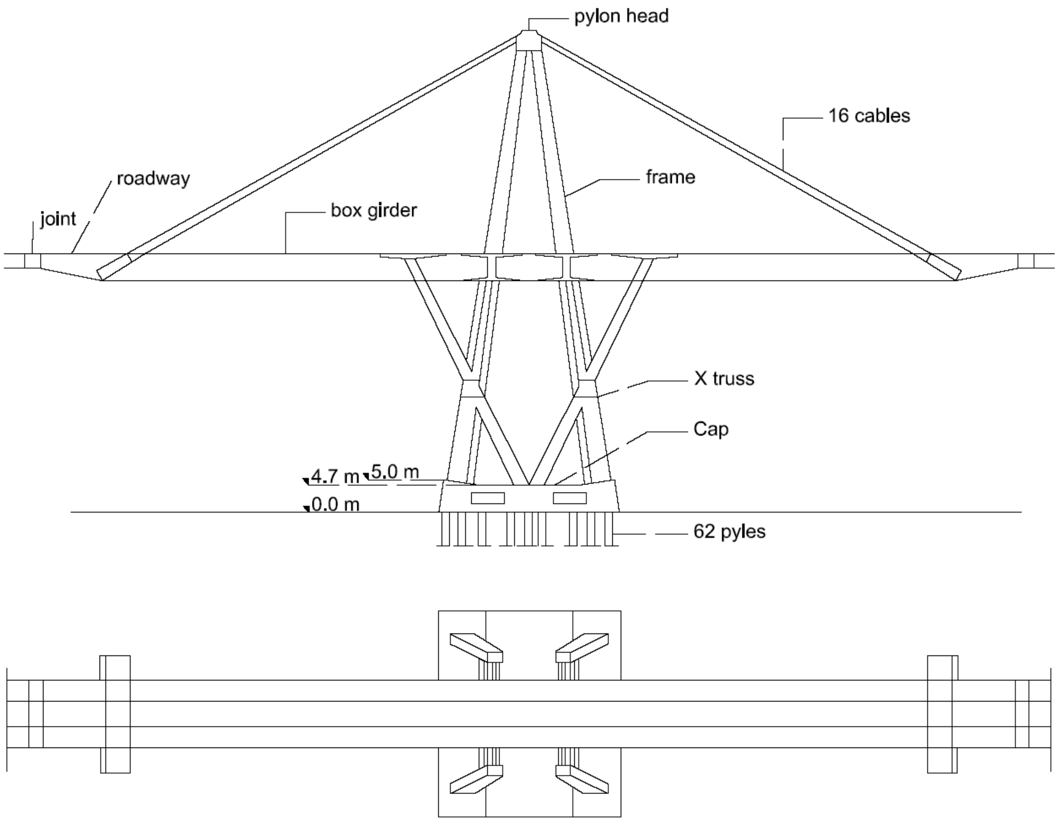

Figure 6. Main span tower and X frames, design scheme

After completion of the pier, service girders are raised into position to be used in the construction of the cantilever arm. Due to the additional moment, produced during this construction stage by the service girder and weight of the cantilever arm, additional concentric prestressing was required in the pier cap (see Figure 5). To avoid overstressing of the X frames during this operation, temporary horizontal ties were installed and tensioned by hydraulic jacks (see Figures 5 and 6). In the construction of the cantilever special steel trusses (service girder) were used for formwork. They were supported at one end by the completed pier cap and at the other end by auxiliary piers and foundations. The anchorages for the cable stays are located in a 73.8 ft (22.5 m) long prestressed inclined transverse girder. The reinforcing cages for these members were fabricated on shore in a position corresponding to the inclination of the cables. They weighed 60 tons and contained 70 prestressing tendons. The cable stays are housed in thick walled steel pipes which were welded to steel plates at their extremity. A special steel spreader beam was used to erect the fabricated cage in its proper orientation. The suspended spans were composed of four prestressed T sections (see Figure 7) for a grand-total of 528 precast prestressed girders.

Figure 7. The 250 t floating crane lifting on piers the 190 t T-shape girder

2.2 Recent Inspection and Interventions

The main observations dealing with on-site data deals with Portillo et al. (2003): in this study, the first phase of the inspection was to check visually the status of cables, sockets, and dampers; this inspection revealed corrosion in both cables and sockets, and the effect of saline winds caused greater corrosion of the cables located in the north side of the bridge. The sockets showed a great degree of deterioration because of corrosion and also considerable displacement. A significant amount of water was found in most of them suggesting the presence of a surface of internal failure in the sockets, which was confirmed in later inspections carried out after a rainfall. Tension of the cables was also measured with an acceleration sensor in the central section of each cable. This sensor was connected to a portable signal analyzer, whose output was processed for calculation of the fast Fourier transform to obtain the main frequency of each cable. The results of the measurement of the tension can be seen for two groups of cables in Table 1. Differences of up to 30% were found in some of the groups, which suggested the possibility that the settlings in the sockets might have been the main cause for the lack of uniformity of tension of the cables. This fact was confirmed by the presence of two types of marks in the sockets of the cables, discovered during the inspection. The first type are those marks made by the displacement of the cables on the last coat of paint applied in 1992 in the area near the sockets, where sections of cable without paint coincided with the length of cable displaced. The second type was found in the dried grease applied in 1992 inside the sockets, where the internal displacement of the cables left a trace. This fact determined the need to retension in order to achieve approximately equal tension in all the cables of each group and thus be able to control future settling in cables. In addition, it would prevent the difference among the tensions from continuing to increase, which might cause the bridge to lose the horizontal configuration of the roadway. On the other hand, the dampers were found to be in good condition. All the aforementioned observations come from Portillo et al. (2003).

Table 1. Initial and final tension in cables before and after re-tension, Pier 20 (Portillo et al., 2003)

	Group 20 northeast		Group 20 northwest		Group 20 southeast		Group 20 southwest	
	T_0 [kN]	T_f [kN]	T_0 [kN]	T_f [kN]	T_0 [kN]	T_f [kN]	T_0 [kN]	T_f [kN]
Cable								
1	1687,18	1741,37	1681,97	1724,52	1730,37	1741,28	1741,24	1741,24
2	1754,87	1741,18	1749,55	1731,61	1752,02	1719,29	1735,58	1731,30
3	1692,15	1727,34	1732,97	1733,52	1713,67	1733,29	1710,92	1727,25
4	1681,24	1727,17	1719,17	1731,22	1740,72	1730,98	1705,32	1727,01
5	1758,51	1728,00	1878,62	1739,52	1797,01	1725,63	1816,51	1725,67
6	1773,43	1731,53	1799,84	1729,57	1749,98	1725,43	1830,27	1725,47
7	1679,05	1724,89	1670,87	1724,75	1686,71	1726,78	1689,44	1736,05
8	1787,47	1738,04	1711,05	1721,20	1716,27	1724,40	1716,31	1735,93
9	1737,97	1730,35	1757,15	1721,01	1787,26	1726,50	1862,82	1724,36
10	1670,29	1733,33	1670,33	1733,33	1756,78	1737,66	1651,63	1720,77
11	1670,25	1726,85	1678,51	1724,36	1721,49	1735,69	1670,41	1726,54
12	1737,72	1724,13	1678,31	1728,05	1745,84	1727,93	1715,99	1720,73
13	1804,80	1736,06	1755,42	1736,30	1824,17	1735,94	1875,05	1736,22
14	1752,23	1735,86	1717,04	1728,44	1727,58	1722,15	1779,87	1726,31
15	1735,26	1721,68	1701,64	1731,07	1699,98	1723,77	1684,12	1733,38
16	1721,53	1725,31	1702,83	1735,58	1781,60	1723,72	1721,72	1733,22

Table 2. Composition of a bridge rope

From inner layer to outer layer	Type of wires	Number of wires	Diameter/high [mm]	Nominal tensile strength
	King wire	1	4.65 d	Blind wire
1	Round wires	6	4.28 d	1600 MPa
2	Round wires	12	4.28 d	1600 MPa
3	Round wires	18	2.28 d	1600 MPa
4	Wedge shaped wires	26	5.00 h	1500 MPa
5	Wedge shaped wires	32	5.00 h	1500 MPa
6	Full-lock wires	24	6.00 h	1500 MPa
7	Full-lock wires	38	6.00 h	1500 MPa

3. Structural Analysis

Since the bridge had to provide ample shipway for ocean-going tankers to and from the important Venezuelan oil-fields located at the head of the lake, five 235 m centre spans were built. To exclude any possible damage resulting from differential settlement of the bridge piers and towers, or from light earthquake shoks, these central spans like all the others have been designed as statically determined systems. For purposes of constructional economy one main span was divided into a cantilever section, while a suspended span constructed of the same prefabricated members as used for the 85 m and 46.60 m spans bridges the gap. The most economical width thus found was 34.60 m in the centre line of the bridge; since the horizontal clearance was specified as 200 m, the cantilever arm length beyond the pile cap is (200-46)/2=77 m. To avoid an otherwise necessary but economical depth of the cantilevers, in spite of this high amount of cantilevering, the tied cantilever construction method was resorted to. The span cantilevered is therefore supported by inclined ropes suspended from the top of a 95.20 m high tower. Any other system of support would have infringed upon the specified clearance (Bauverlag, 1963). Assumed hinges reduced the highly statically indeterminate system of the piers to a three times indeterminate principal system, namely two hinged frame with latticed legs and additional support of the cantilever arms: in this simplified hinged system the intersecting forces were firstly computed by the method of elimination of geometrical elements. Torsions of the junctions in the hinged system were the basis for computing in a second operation the moments of constraint in the pier legs and in the tower by the deformation method; in this way the actually flexurally-rigid junctions were taken into account as such. It should be considered also that because of the great rigidity differences from these secondary forces were small if compared to the main forces computed for the simplified hinged system: therefore the influence computed for the hinged system were revealed to be in accordance with reduced model produced before construction, carried out at the National Laboratory of Civil Engineering of Lisbon. The cantilever span rests on X-frames: the relatively short distance between the legs of such a frame compared with the cantilever length results in almost rigid restraint. The cantilever span is supported by inclined ropes suspended from the top of a 92.50 m high four-legged structure of two inclined A-frames linked at the top to a transverse girder. The four legs of this tower are nowhere connected with either the cantilever span or with the X-frames. Concerning the continuous cantilever girder, this structure is represented by a closed box section 5 m deep: its torsional stiffness effectively distributes unsymmetrical traffic loadings; in addition, the high natural torsional frequency of the box section, avoids resonance with the natural frequency of the ropes. Since strains of the ropes were equalized during construction, the intersecting forces of the system resulting from deadweight were calculated on the assumption of infinitely rigid inclined ropes. For the traffic loads however elasticity of the ropes had to be taken into account. Due to the relatively flat slope of the ropes, an axial force was induced into the cantilever girder allowing mild steel reinforcement in nearly all parts of it. Additional tendons were only required in the girder above the X-frames and the prestressed transverse girders for anchoring the suspension ropes in order to account for moment raisers. Moreover, a structural analysis for the various stages of construction had been realized. In view of the weight of the structure, each main pier and tower of the long spans rests on sixty-two inserted piles flexurally-rigidly capped by a heavy concrete slab of 34.60 m wide in the centre line of the bridge and 39 m in the transverse direction, with a thickness of 4.7 m over the central section sloping up to 5.90 m at the edges. One pile cap contains 5.100 m^3 of B300 concrete and 400 t of reinforcing steel. Again, the reinforcing concrete bottom shuttering was cast on shore, and each shuttering unit of a pile cap required twenty precast slabs of 50-60 t each. The different lengths of the piles were equalized by varying the lengths of the suspensions for the precast slabs. The pile cap had to pour in five layers in order to avoid local overstressing. Two floating plants of 40 m^3/h capacity each were used for concrete fabrication and pouring. The X-frames are of double T-girder section: while having an equal width of 5.25 m from bottom to top transversely to the centre line of the bridge, the width in the direction of the bridge tapers from 2.57 m at the bottom to 1.38 m at the top. Two adiacent X-frames are tied at their junctions by a cross beam. Climbing shutters of he Luchterhand type were used for the X-frames: the reinforcing cages were again fabricated on-land in a length of 9.5 m and placed into position by a tower crane. Concrete type B450 was used for X-frames and towers. The structural analysis realized at that time for the various conditions showed that during concreting the inclined legs had to be supported at a definite spacing so as to keep stress and deformations within design limits. Before concreting the third section, inclined braces were installed and preloaded up to 20 t by 100 t hydraulic jacks. Higher up, the outer braces were tied by members each composed of six prestressing rods. Rod diameter was 26 mm and ultimate strength of steel was from 80 to 105 kg/mm^2; each rod was prestressed by a 35 t jack to the statically required load. The rod rested on tubular centering to avoid vibration. These stresses on rods were continuously checked and adjusted by the jacks. Each tower is a four-legged structure of two inclined A-frames linked at the top by a transverse girder, 92.5 height. The cross section changes with increasing height from 4.96 m by 2.20 m at the bottom to 2.31 m by 2.92 m at the top. This unusual shape complicated shuttering and reinforcement and called for close checking of the alignment. Therefore, it was not possible to resort to the shuttering method used for the X-frames. For the towers, the shuttering had to be a box type with guide frames, so that its shape could continuously be adjusted by screw jacks to the variation of the tower cross-section. Like the X-frames, the inclined tower legs also had to be braced during construction: to facilitate this bracing, the X-frames were erected sufficiently ahead of the towers so that they could be used for bracing the tower (Bauverlag, 1963).

4. Lifetime Coupled Assessment

The coupled assessment of the structure will be focused on the bridge ropes, the most prone detail to the coupled damage. These were made with cold-drawn patented steel wires (Bauverlag, 1963): the thirty-two ropes of a single system have to carry a permanent load of 5.425 t per rope plus approximately fourteen tonns of traffic load; the breaking strength of a rope was determined as a result of four test at 601 t. The ropes were cut to proper length and their conical ends poured in-shop. The conical ends had internal threads to accept the stretching screws: these ends were tested for tight fit by elongation and long-time tests. The composition of a single bridge rope is reported in Table 2. Load models have been implemented according to EN 1991-2 (2005), in detail the load fatigue model 1 (LFM1). First permanent loads acting on the bridge and then those induced by traffic are included according to conventional lanes. Finally the combinations of loads at the fatigue limit state are investigated, according to EN 1991-2 (2005). The fatigue endurance of tension components has been determined using the fatigue actions previously described and the appropriate category of structural detail. The effective category should preferably be determined from tests representing the actual configuration used. However, in the absence of the tests described above, fatigue strength curves according to EN 1993-1-11 (2007) may be used. In this work three different cables have been considered with the aim of obtaining some elements about fatigue design for this particular case. So, fatigue verification has been performed according to EN 1993-1-9 (2005) procedures, according to the following detail category: C= 160 for parallel wire strands with epoxy socketing, C=150 for spiral strands with metal or resin socketing and C=105 for prestressing bars. These categories are intended to be applied in compliance with the general requirements, including the preliminary check for stress limit of EN 1993-1-11 (2007). Load models have been checked for the maximum stress oscillations induced in the cable groups, according to EN 1-2 (2005): load model considered are all the fatigue load model presented in Eurocodes (FLM1-FLM4). Additionals fatigue load models have been introduced in order to check for peak level oscillations, dealing with not-frequent but high-peak traffic induced situations, accounted by the characteristic values reported in the distributed load of FLM1: fatigue model 5 deals with an unbalanced load in the two ways (one completely full, the other without traffic); fatigue model 6 deals with an unbalanced load in the two bridge segment parts (the north side completely full, the south side without traffic). Stress oscillations obtained by this analisys are reported in the following table (Table 3).

Table 3. Peak cable stress oscillations

Cable 1-16	Group 20 northeast	Group 20 northwest	Group 20 southeast	Group 20 southwest
	[MPa]	[MPa]	[MPa]	[MPa]
Fatigue load model 1	60,13			
Fatigue load model 2		107,05		
Fatigue load model 3			100,76	
Fatigue load model 4				95,38
Fatigue model 5	70,94			
Fatigue model 6			111,66	

Fatigue damage results are reported in Table 4: only parallel wire strands (C=160) resulted to be verified; in particular, all other sets of combination category detail, resulted to be affected by insufficient fatigue strength in correspondence with different fatigue models, even if also C=150 is verified against fatigue except for the last load model type. The partial conclusion is that the suspension system must be protected against corrosion by metallic and/or organic coatings if used in any aggressive environment: moreover, according to recent studies (Nürnberger, 2007; Lan & Li 2009; Lan et al., 2009) the formation of a fatigue crack is influenced by physical-chemical interactions between the environment and the steel surface, activated by fatigue. Not only liquids, but also gases and vapours may accelerate the deterioration process. Dry air is already a surface-active medium and reduces the fatigue strength in comparison to the vacuum. Coatings impermeable to oxygen and steam (e. g. sufficiently thick reactive resins) improve the fatigue behavior not only in corrosive environment, but also in the air. In the present situation, an improvement could be reached for example by protecting the existing wire. Other concurring fatigue detrimental facts could influence the lifetime of the suspension system, as out of round sheaves, tight grooves, misaligned sheaves, undersized sheaves, worn bearings, vibration, slapping, whipping, reverse bends. Whereas adequate protection against corrosion is no more provided, the fatigue strength of cables could be compromised (Funahashi, 1995; Sih et al., 2008; Virmani, 1993). In order to give some insights on the effect of corrosion and concurring detrimental factors on the fatigue strength of cables, different corrosion propagation rates have been simulated. The corrosion rates have been considered according to

Elachachi et al. (2006), Dieng et al. (2009), Cremona (2003), Weischedel and Hoehle (1995) and Camo (2003), checking their influence on the fatigue strength of the single cable. In Fig. 8 different possible propagation rates (P.R.) have been assumed: P.R.1 stands for a linear effective area reduction of 10% in 60 years; P.R.2 for 25% in 60 years; P.R.3 for 50% in 50 years; P.R.4 stand for 60% in 20 years. The main results of this investigation have been reported respectively in Table 4, where the corrosion limit is defined according to the maximum resisting area needed to satisfy the fatigue verification for the acting loads.

Table 4. Fatigue verification, according to Eurocode (1993-1-9, 2005)

Load model type	Cable	Peak group stress oscillation [MPa]	Detail category [MPa] Prestressing bars	Damage equivalent fatigue verification
Fatigue load model 1	Cable 1-16	60,13	105	OK
Fatigue load model 2	Cable 1-16	107,05	105	NO
Fatigue load model 3	Cable 1-16	100,76	105	NO
Fatigue load model 4	Cable 1-16	95,38	105	NO
Fatigue load model 5	Cable 1-16	70,94	105	OK
Fatigue load model 6	Cable 1-16	111,66	105	NO
Load model type	Cable	Peak group stress oscillation [MPa]	Detail category [MPa] Spiral strands with metal or resin socketing	Damage equivalent fatigue verification
Fatigue load model 1	Cable 1-16	60,13	150	OK
Fatigue load model 2	Cable 1-16	107,05	150	OK
Fatigue load model 3	Cable 1-16	100,76	150	OK
Fatigue load model 4	Cable 1-16	95,38	150	OK
Fatigue load model 5	Cable 1-16	70,94	150	OK
Fatigue load model 6	Cable 1-16	111,66	150	NO
Load model type	Cable	Peak group stress oscillation [MPa]	Detail category [MPa] Parallel wire strands with epoxy socketing	Damage equivalent fatigue verification
Fatigue load model 1	Cable 1-16	60,13	160	OK
Fatigue load model 2	Cable 1-16	107,05	160	OK
Fatigue load model 3	Cable 1-16	100,76	160	OK
Fatigue load model 4	Cable 1-16	95,38	160	OK
Fatigue load model 5	Cable 1-16	70,94	160	OK
Fatigue load model 6	Cable 1-16	111,66	160	OK

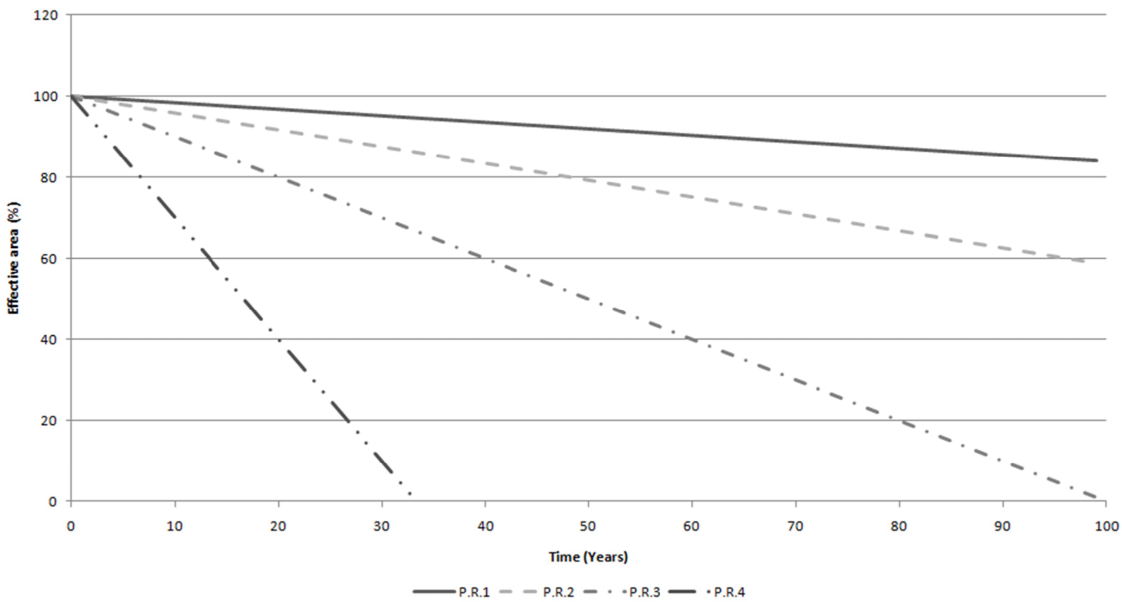

Figure 8. Corrosion propagation, different increasing rate (P.R.i)

For instance, an area loss of 21.1% for the FLM1, with the C=105 (Table 5) imply an insufficient fatigue strength of the cable gropus analized. Common effective area reductions have been observed for example by Mayrbaurl (2000) and Parag et al. (1999). Moreover, the consequent lifetime of cables under the aforementioned propagation rate (P. R.) is reported in Table 6 in time-years. In this table, the results in terms of lifetime for the various models analized and the four propagation rates are shown: it should be observed that the lifetime assessment is achieved for the whole fatigue model only considering C=160, parallel wire strands with epoxy socketing; if assuming C=150, spiral strands with metal or resin socketing, only a few FLM are verified; finally, considering the C=105, prestressing bars, only a two FLM are verified. As a consequence, considering that the bridge design working life is typically considered 100 years, according to Eurocode (EN 1990, 2006), the optimal design configuration should be chosen also according these considerations.

Table 5. Corrosion limit

Load model type	Cable	Peak group stress oscillation [MPa]	Detail category Prestressing bars [MPa]	Corrosion limit [%]
Fatigue load model 1	Cable 1-16	60,13	105	78,09
Fatigue load model 2	Cable 1-16	107,05	105	-
Fatigue load model 3	Cable 1-16	100,76	105	-
Fatigue load model 4	Cable 1-16	95,38	105	-
Fatigue load model 5	Cable 1-16	70,94	105	92,14
Fatigue load model 6	Cable 1-16	111,66	105	-
Load model type	**Cable**	**Peak group stress oscillation [MPa]**	**Detail category Spiral strands with metal or resin socketing [MPa]**	**Corrosion limit [%]**
Fatigue load model 1	Cable 1-16	60,13	150	55,16
Fatigue load model 2	Cable 1-16	107,05	150	98,21
Fatigue load model 3	Cable 1-16	100,76	150	92,44

Fatigue load model 4	Cable 1-16	95,38	150	87,50
Fatigue load model 5	Cable 1-16	70,94	150	65,09
Fatigue load model 6	Cable 1-16	111,66	150	-
Load model type	Cable	Peak group stress oscillation	Detail category Parallel wire strands with epoxy socketing	Corrosion limit
		[MPa]	[MPa]	[%]
Fatigue load model 1	Cable 1-16	60,13	160	51,83
Fatigue load model 2	Cable 1-16	107,05	160	92,28
Fatigue load model 3	Cable 1-16	100,76	160	86,86
Fatigue load model 4	Cable 1-16	95,38	160	82,22
Fatigue load model 5	Cable 1-16	70,94	160	61,16
Fatigue load model 6	Cable 1-16	111,66	160	96,26

Table 6. Lifetime analysis

Model type C=105	Cable	Lifetime P.R.1 [years]	Lifetime P.R.2 [years]	Lifetime P.R.3 [years]	Lifetime P.R.4 [years]
Fatigue load model 1	Cable 1-16	>100	52	22	7
Fatigue load model 2	Cable 1-16	-	-	-	-
Fatigue load model 3	Cable 1-16	-	-	-	-
Fatigue load model 4	Cable 1-16	-	-	-	-
Fatigue load model 5	Cable 1-16	50	19	8	4
Fatigue load model 6	Cable 1-16	-	-	-	-
Model type C=150	**Cable**	**Lifetime P.R.1 [years]**	**Lifetime P.R.2 [years]**	**Lifetime P.R.3 [years]**	**Lifetime P.R.4 [years]**
Fatigue load model 1	Cable 1-16	>100	>100	45	15
Fatigue load model 2	Cable 1-16	11	4	2	1
Fatigue load model 3	Cable 1-16	50	19	8	3
Fatigue load model 4	Cable 1-16	81	31	13	4
Fatigue load model 5	Cable 1-16	>100	84	35	12
Fatigue load model 6	Cable 1-16	-	-	-	-
Model type C=160	**Cable**	**Lifetime P.R.1 [years]**	**Lifetime P.R.2 [years]**	**Lifetime P.R.3 [years]**	**Lifetime P.R.4 [years]**
Fatigue load model 1	Cable 1-16	>100	>100	49	16
Fatigue load model 2	Cable 1-16	48	19	8	3
Fatigue load model 3	Cable 1-16	81	31	13	4
Fatigue load model 4	Cable 1-16	>100	43	18	6
Fatigue load model 5	Cable 1-16	>100	93	39	13
Fatigue load model 6	Cable 1-16	25	9	4	2

5. Conclusions

The fatigue life of a cable-stayed bridge can be improved by seeking the most efficient construction details, and optimizing the design of the suspension system in order to minimize live load stress ranges and lifetime detrimental effects. This study has been carried out in order to deep the lifetime assessment of existing cable bridges, evaluating the concurrent effect of the coupled damage due to fatigue and environmental damages. The study carried out, allows evaluation of some problems that may originate in the cables of an existing cable-stayed bridge, specifically with respect to cable type and waterproofing. The rather significant force generated from fatigue loadings may cause localized failures, especially if the deterioration due to corrosion processes is still in progress. The analysis presented in this paper aims to suggest a specific procedure to be carried out from the design stage of cable stay and/or supported bridges, in order to check for their suitable use during the whole lifetime of the bridge structure, minimizing the repair costs needed for sub-structure modifications or replacement, that could also negatively influence the utilization of the same structure. The particular case study analyzed is an iconic bridge, in service from 50 years of service life. After presenting the case study considered, a fatigue and lifetime assessment has been carried out: the fatigue strength resulted to be checked by the use of parallel wire strands with epoxy socketing, for the whole fatigue load model considered. Then, lifetime corrosion assessment has been studied in accordance to various propagation rates coming from literature, and the critical resisting area loss of cables has been found for the different models analized. As a fact, this study is based on literature data (Portillo, 2003) on an European standard procedure both for loadings (EN 1991-2, 2005) and for fatigue verifications (EN 1993-1-9, 2005).

In order to check for a definitive assessment, the structure should be inspected in all of its components and sub-structure, monitored for real observed traffic data, and tested at least with a precise NDT cycle of analysis. Once more specific data are available, a further step considering the theoretical basis for assessing the safety of a specific type of cables should be formulated considering the safety in regard to both the ultimate capacity as well as the effects of fatigue and corrosion. Moreover, probabilistic models could be adapted to experiments results obtained by testing standard wire specimens in the laboratory. Then, inspections of the condition of the wires of a cable may be used to update the reliability of a cable group. By modelling the quality of the performed inspection method in terms of its ability to detect wire ruptures the reliability of the cable as a whole may be updated. As a matter of fact, the aspects of design of cables have been addressed and it is shown that the effect of degradation plays an important role for the safety and thus the design of hanger and stay cables.

For this reason, this study aims only to represents a first approach to the issue, a method to be applied to real case structure, to be confirmed for specific case by more deepen and on-site analysis.

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