# Fatigue assessment of composite steel-concrete cable-stayed bridge decks

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**Abstract**

Fatigue safety verification is an important part in the steel highway and railway bridge design. The part 2 of the Eurocode 3 (EC3-2) proposes a simple and fast fatigue verification procedure. This one consists to determine a value of an equivalent stress range based on the passage of the vehicle FLM3, which is multiplied by a λ factor, called damage equivalent factor, and to compare it with the resistant stress range of each selected fatigue detail.

However, λ factor has limits and it is not defined in the EC3-2 for some forms and lengths of influence lines. Cable-stayed bridges are precisely included in fields in which this procedure is not effective.

The objective of this Master thesis is to obtain the damage equivalent factor λ for cases, which are not valid in the EC3-2. In this content, an adjustment of the fatigue verification procedure will be proposed in order to structural systems such as cable-stayed bridges are taken into account.

**Key-words**: Cable-stayed bridge; Fatigue design; Fatigue load model; Influence line; Damage equivalent factor.

## 1. Introduction

Cable-stayed bridges are new and elegant structures. For the last 30-40 years, construction of cable-stayed structures has been developed rapidly with span record and important technological advances and today it is considered as the most modern structural system for bridge. Nowadays, concrete and steel, the two most popular materials in the constructions, are used in an optimal way to have more economic structures (Virlogeux, 2002) [1].

A combination of two internal forces in the deck is the main characteristic of the cable-stayed structure: flexion and compression, which is induced by the stays.

This project will focus on the fatigue verification procedures for cable-stayed bridge. The main procedure described in the EN1993-2 (EC3-2, 2006) [2] is the damage equivalent factor method. This method is based on a parameter, noted λ factor, which depends on the critical length of the influence line loaded. However, this λ factor is not calibrate for critical length higher than 80 m.

Then influence lines of cable-stayed bridge may be very complex and can have critical lengths much higher than 80 m. Indeed, as explained previously, cable-stayed system is a structural system composed by two internal forces: bending moment and axial force. These two forces involve two different influence lines and it is not clear which one is the best to describe the maximum and minimum stresses. Thus, stress influence lines must be defined to solve this problem in order to combine both of influence lines.

## 2. Fatigue design

### 2.1. Fatigue procedure

Fatigue verification procedures are similar to the structural verifications and consist to verify that all traffic load effects are lower than the resistance of the bridge. The damage equivalent factor method is described in the article 9 of the EN1993-2 [2] as follows:

$$\gamma_{Mf} \cdot \gamma_{Ff} \cdot \Delta\sigma_{E2} \leq \Delta\sigma_C \quad (1)$$

$$\Delta\sigma_{E2} = \lambda \cdot \Phi_2 \cdot \Delta\sigma \quad (2)$$

$\Delta\sigma_{E2}$ is the damage equivalent stress range at 2 x $10^6$ cycles and must be calculated with the damage equivalent factor λ. $\Delta\sigma_C$ is the given stress according to the selected fatigue detail and is also called the FAT value. This value is categorized in the tables 8.1 to 8.10 of the EN 1993-1-9 (EC3–1-9, 2005) [3]. Then, $\Phi_2$ represents the damage equivalent impact factor and may be taken as equal to 1.0 for road bridges. Finally, two partial safety factor must be taken into account and are:

- $\gamma_{Mf}$ for the fatigue action effects and is equal to 1.0;
- $\gamma_{Ff}$ for the fatigue strength and is equal to 1.35 in this project, as recommended in the table 3.1 of the EN 1993-1-9 [3].

The damage equivalent factor $\lambda$ can be calculated according to the article 9.5.2 in the EN 1993-2 [2], as follows:

$$\lambda = \lambda_1 \cdot \lambda_2 \cdot \lambda_3 \cdot \lambda_4 \leq \lambda_{max} \quad (3)$$

It is the product of four partial factors to take into account characteristics such as the composition and volume of the traffic or the working life of the bridge. A limit was also put with the factor $\lambda_{max}$ that represents the maximum damage equivalent value and allows to avoid that the multiplication of the individual partial factor may result in a value far exceeding the one obtained from a design using fatigue limit (ECCS, 2011) [4]. This maximum value depends of the critical length of the influence line ($L_{crit}$) and the type of section. The first partial factor $\lambda_1$ represents the damage effect of traffic and depends on the critical length like $\lambda_{max}$. The second one, $\lambda_2$, is the factor for the traffic volume and should be calculated by:

$$\lambda_2 = \frac{Q_{m1}}{Q_0}\left(\frac{N_{obs}}{N_0}\right)^{1/5} \quad (4)$$

The partial factor $\lambda_3$ is used for the design working life of the bridge, taking into account the design life in year with the parameter $t_{Ld}$. A design lifetime of 100 years has been chosen to obtain $\lambda_3$ = 1.0. Finally, $\lambda_4$ represents the traffic on other lanes and considers particularly the number of heavy traffic per year and the average weight of them and is equal to 1.0 too.

The $\lambda$ factor is obtained by using the method of Hirt, as showed in the Figure 1. This one consists to the division between the stress variations due to a fatigue load model, usually FLM3, and the ones due to a "real traffic".

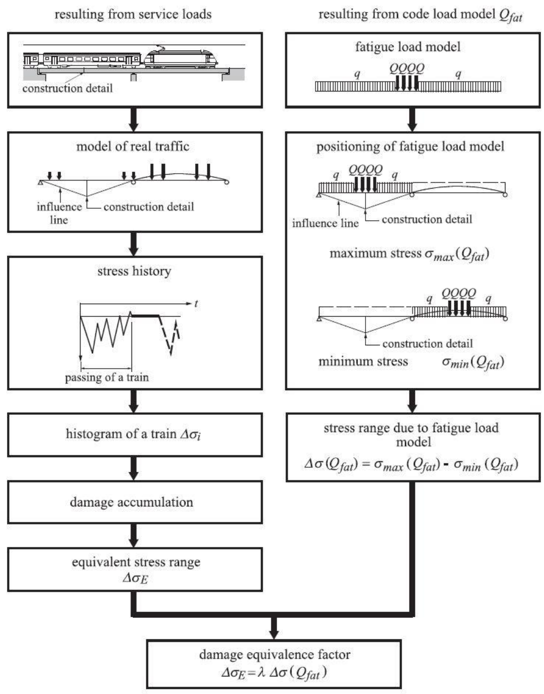


*Figure 1 : Method of Hirt [4]*

Usually associated to the equivalent stress range at 2 x $10^6$ cycles and to the damage equivalent factor method, the fatigue load model 3 is used and that means it is a very important model for engineers. This is simple model of a single vehicle with 4 axles of 120 kN each for a total weight of 480 kN and its geometry is shown in Figure 2.

Then, according to the EN 1991-2 (EC1-2, 2003) [5], a second vehicle should be taken into account if it is relevant. The geometry of this second vehicle is the same as the first one with a reduced weight of 36 kN, instead of 120 kN, per axle and a minimum distance of 40 meters between the two vehicles.

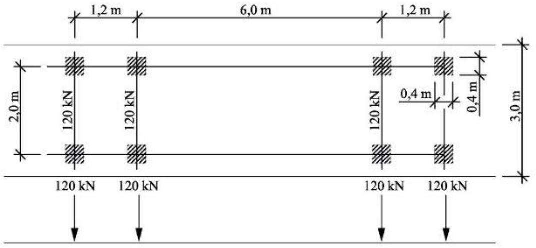


Figure 2 : Fatigue load model 3 [4]

### 2.2. Fatigue curves

As explained previously with the eq. (1), a fatigue detail must be selected to define a FAT value using the Eurocodes. To do so, standard curves, also called S-N curves, have been created for different fatigue details. These curves are useful to verify that stress variation is lower than the limit and have also been determined with fatigue tests in which specimens are subjected to repeated cyclic loading with a constant stress range.

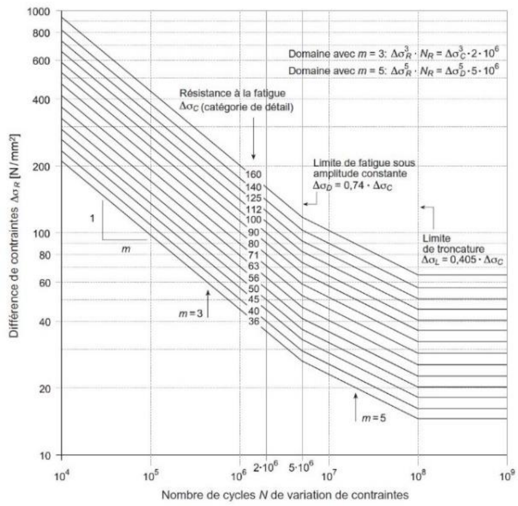


Figure 3 : Fatigue curves for steel elements (TGC 10, 2001) [6]

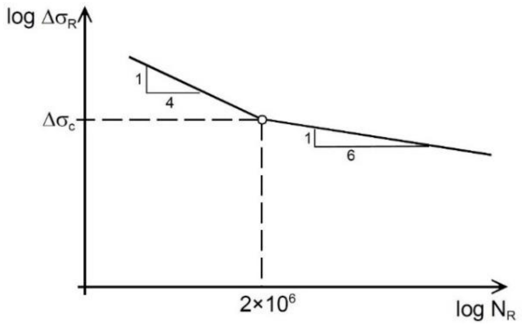


Figure 4 : Fatigue curves for tension components (EC3-1-11, 2006) [7]

These curves are showed in the Figure 3 and Figure 4, with the number of cycle (*N*) on the abscissa and the stress range (*Δσ*) on the ordinate. After some researches about stays, it has been demonstrated that fatigue strength for them have a different behaviour than other steel elements and that is why there are two types of fatigue curves. Thus, there is one fatigue curve for each detail category and these curves are also described with the following expression:

$$N = C \cdot \Delta\sigma^{-m} \qquad (5)$$

where *m* is the slope coefficient and *C* is a constant representing the influence of the structural detail.

## 3. Study case

This study case is based on the *Vasco da Gama* bridge and the main characteristics are taken from the PhD thesis of the Professor José J. Oliveira Pedro (Pedro, 2007) [8].

The longitudinal configuration is a 2D-model and shows two lateral spans of 204.5m and a central span of 420m for a total of 829m in length. This model considers two towers and three piers (three in each side span, which prevent excessive flexion in the towers. Finally, the whole deck is supported by two couples of 16 stays for a total of 64 stays. The towers and the piers are in concrete and the deck is a composite steel-concrete one.

This deck is composed by two longitudinal I-shape steel girders with a height of 2.25m and longitudinal and transversal stiffeners and transversal girders spaced of 4.375m. The concrete part is composed by precast concrete slab panel with a thickness of 0.25m. The connection between steel and concrete are insured with studs. Moreover, the slab's armatures are ignored in this model and, as a 2D-model, it considers the half of the bridge's width into consideration with an effective concrete deck width of 7.5m ($b_{eff}$ = 7.5m).

The stays are directly linked to the main steel girders and are spaced with a distance of 13.125. For facilities, all the stays are numbered from L1 to L16 for lateral span and from C1 to C16 for central span, starting with the closest

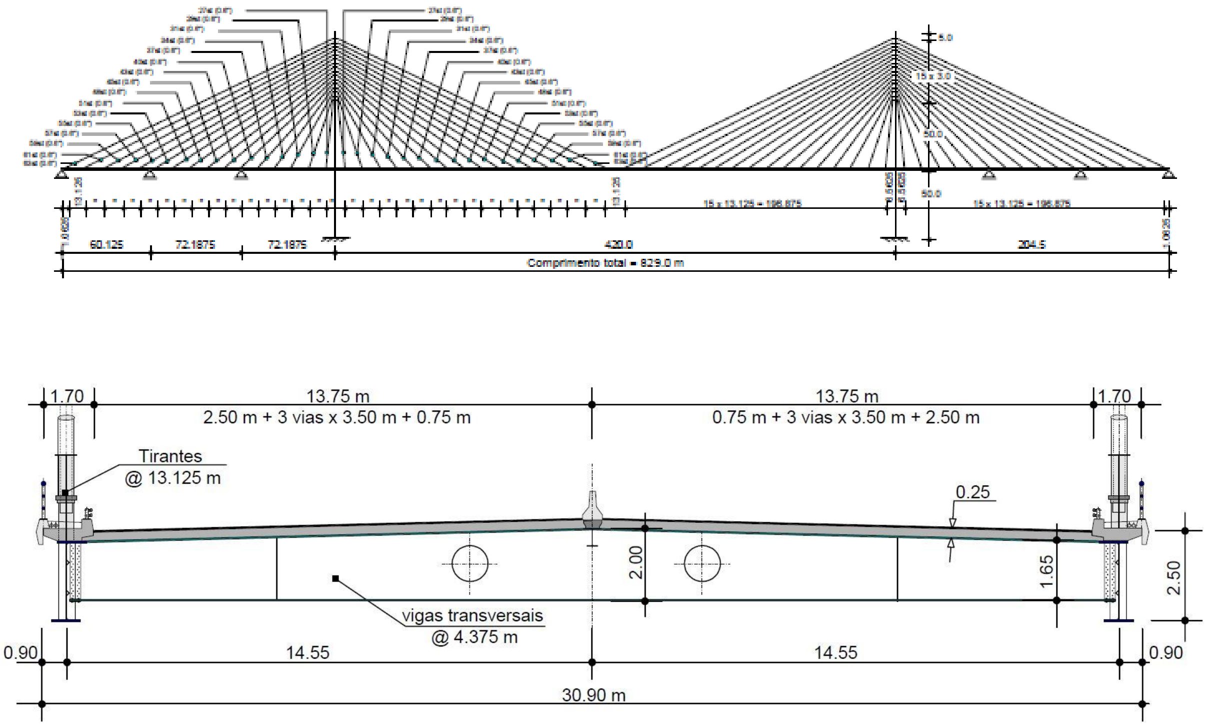


*Figure 5 : Case study details [8]*

one of the tower. The dimensions vary between the first stay (near to the tower) with a diameter of 27st (0.6’’) to the last stay with a diameter of 63st (0.6’’). For information, 27st (0.6’’) means 27 strands T15, which is equal to 27 * 150mm$^2$.

## 4. Fatigue details

Regarding the fatigue details of stays, we must analyse first their anchorage. The Figure 6 shows that the anchor is directly welded to the main steel girder. This anchorage is composed by a steel sheet with two stiffeners. The stay is then put in the available space and fixed with a ring.

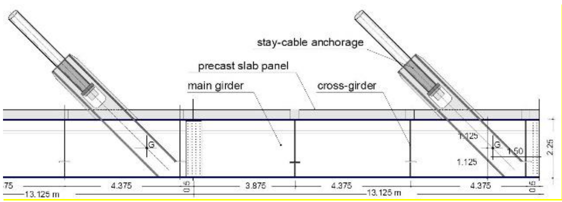


*Figure 6 : Details of stays anchorage*

So, there are a lot of different details but in order to seek simplicity, the selected detail is the stay’s breaking close to the anchorage. This detail is not directly important in terms of the fatigue but it allows to use some simplifications for the next calculations such as to only have axial forces in the stay. According to the EN 1993-1-11 [7], this detail has a FAT value of 160 MPa from the fact that the stays are made of strands.

## 5. Influence lines

The influence line allows to define the stress range in one element in a specific location under mobile load. Using a unitary mobile load allows to multiply the influence line’s curve by the value of the total action of the vehicle to obtain the desired stress ranges. Thus, using influence line turns into a great advantage.

As explained previously, cable-stayed systems is composed by two internal forces which involve two different influence lines. To determine the extreme stresses locations, stress influence lines are used and they are defined by the next expression:

$$\Delta\sigma_B = \frac{\Delta M}{W} + \frac{\Delta N}{A} \tag{6}$$

where W is the section modulus value and A is the section area of the element. Considering the selected fatigue detail for stays, there is no bending moment and thus, the eq. (6) can be simplified.

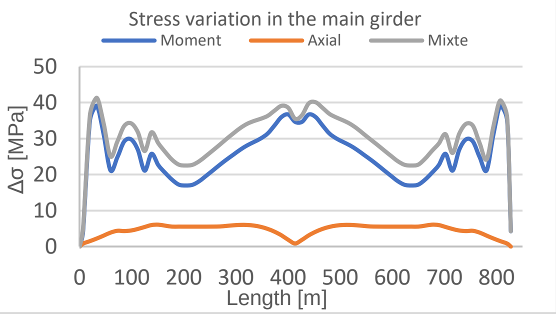


Figure 7 : Stress variation in the main girder

However, as showed in the Figure 7, the axial stress has a very little impact on the total one when the bending stress is very similar to the total stress of the main girder. In this fact, it has been decided for fatigue details with two internal forces to use the maximal variation of the bending stresses to determine the total stress and add the associated normal stresses, even if it is not the maximum and minimum.

Using the case study described, two types of stress influence lines are defined according to the lateral and central stays. As showed in the Figure 8 and Figure 9, lateral stays are irregular with complex stress influence lines. That involves that the critical lengths are difficult to determine. At the opposite, central stays are more simple with regular and simple shape.

Moreover, influence lines allow to define the critical length which is an important parameter in the damage equivalent factor procedure. It is possible to determine this length according to the influence line's type. Using the article 9.5.2 of EN 1993-2 [2], the critical length may be defined for simple influence lines. In the Table 1, all stays information are summarized.

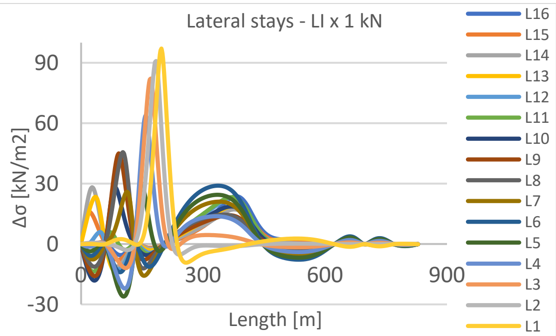


Figure 8 : Stress influence lines of lateral stays

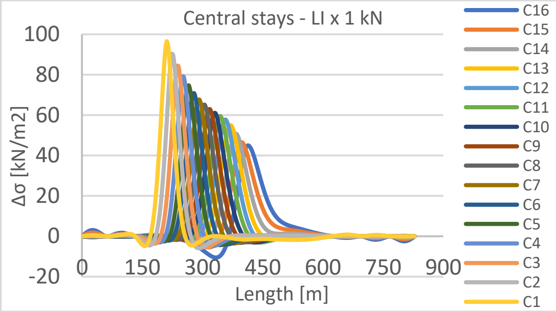


Figure 9 : Stress influence lines of central stays

| N° | Force - Section | $L_{crit}$ [m] |
|---|---|---|
| L1 | Moment - Midspan | 89 |
| L2 | Moment - Midspan | 100 |
| L3 | Moment - Midspan | 81 |
| L4 | Moment - Support | 315 |
| L5 | Moment - Support | 315 |
| L6 | Moment - Support | 145 |
| L7 | Moment - Midspan | 71 |
| L8 | Moment - Midspan | 71 |
| L9 | Moment - Midspan | 71 |
| L10 | Moment - Midspan | 71 |
| L11 | Support | 100 |
| L12 | Moment - Support | 140 |
| L13 | Shear - Midspan | 54 |
| L14 | Shear - Midspan | 54 |
| L15 | Shear - Midspan | 54 |
| L16 | Moment - Support | 135 |
| C1 | Moment - Midspan | 89 |
| C2 | Moment - Midspan | 90 |
| C3 | Moment - Midspan | 90 |
| C4 | Moment - Midspan | 90 |
| C5 | Moment - Midspan | 105 |
| C6 | Moment - Midspan | 100 |
| C7 | Moment - Midspan | 100 |
| C8 | Moment - Midspan | 100 |
| C9 | Moment - Midspan | 125 |
| C10 | Moment - Midspan | 130 |
| C11 | Moment - Midspan | 145 |
| C12 | Moment - Midspan | 150 |
| C13 | Moment - Midspan | 150 |
| C14 | Moment - Midspan | 160 |
| C15 | Shear - Midspan | 162 |
| C16 | Shear - Midspan | 162 |

Table 1 : Critical lengths for stays

One can notice that the lateral stay 11 (*L11*) has no force's type. This one represents the stay linked to the second pier (*P2*) and has a very complex shape, which was impossible to compare with the simple influence lines of the EN 1993-2 [2]. It has been also decided to take a support section with a critical length equal to 100m because it is the most unfavourable value for the λ factor.

## 6. Fatigue assessment

To perform the damage equivalent factor procedure on the stays, the λ factors must be defined. However, the λ factors are not defined for critical lengths higher than 80 m. Hypotheses should be done. To do so, they are based on the PhD thesis of Nariman Maddah (Maddah, 2013) [9] and the results he obtained. These results are based on the Swiss traffic with $N_0$ = 500'000 heavy vehicles per year using FLM4 with traffic type of *long distance* and are showed in the Figure 10.

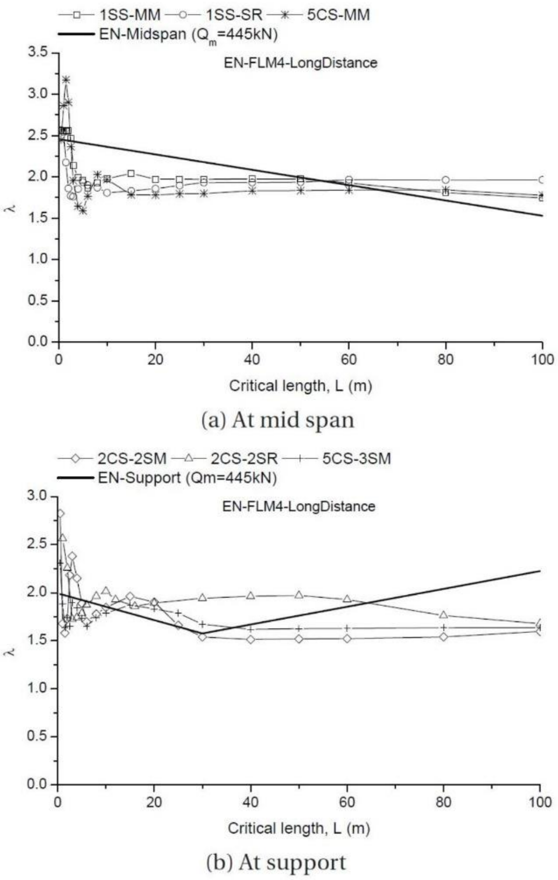


*Figure 10 : Comparison of Eurocode damage equivalent factor with FLM4 for long distance traffic [9]*

One observes that the trend for critical length between 80m and 100m is constant for a midspan section or a section at support. It has been then decided to keep the value of the partial factor $\lambda_1$ constant for critical lengths higher than 80m, written as follows:

$\lambda_1$ → = 1.85 (midspan) &
= 2.20 (support)

The values of the other partial factors are:

$\lambda_2 = 1.22$ $\lambda_3 = 1.00$ $\lambda_4 = 1.00$

The results for some stays are presented in the following table.

| | | L1 | C1 | C16 |
|---|---|---|---|---|
| Force | | Bending | Bending | Shear |
| Section | | Midspan | Midspan | Midspan |
| $L_{crit}$ | [m] | 100 | 90 | 162 |
| $\lambda$ | [-] | 2.26 | 2.26 | 2.26 |
| $\lambda_{max}$ | [-] | 2.00 | 2.00 | 2.00 |
| $\Delta\sigma_{E,2}$ x 1.35 | [MPa] | 137.57 | 131.55 | 71.93 |
| $\Delta\sigma_C$ | [MPa] | 160.00 | 160.00 | 160.00 |
| Verification | | OK | OK | OK |

*Table 2 : Fatigue verification with FLM3*

One will first notice that all critical lengths are higher than 80m. That involves that the values of *λ* factor vary between 2.26 for midspan. These values are close or higher to the maximal limit according to the Eurocode ($\lambda_{max}$).

Then, all the selected stays satisfy the fatigue verification. However, this is not enough to confirm that the constant trend hypotheses are corrects. Indeed, it will be wise to compare these results with those obtain from a "real traffic".

## 7. Damage equivalent factors

To determine new *λ* factors for critical lengths higher than 80m, the method of Hirt has to be performed. This method is based on the Figure 1 and consists of calculating a stress range with a load model (usually FLM3) and an equivalent stress range at 2 x $10^6$ cycles with a total damage of 1.0, using a "real traffic" with FLM5. The division between these two stresses gives the damage equivalent factor, as described in the following relation:

$$\lambda = \frac{\Delta\sigma_{E,2}}{\Delta\sigma(\mathrm{Q_{fat}})} \tag{7}$$

First of all, the stress range according to the model must be calculated. The model used is FLM3 (Figure 2), based on the EN 1991-2 [5]. This stress range is determined using a main lorry with a total load of 480 kN and a second lorry with a load of 144 kN (4 axles of 36 kN instead of 120 kN) at a distance of 40m. The second vehicle has the same geometry of the main one.

For the "real traffic", it has been decided to generate a traffic based on the lorries of the FLM4. Thus, the model used is a kind of simplified FLM5 composed by six different vehicles, which are a normal vehicle with a load of 0 kN and the five lorries of FLM4 with the associated load. Moreover, it has been also decided to consider 25% of the heavy vehicles in the traffic.

$Q_0$ = 0 kN $\quad$ $Q_1$ = 200 kN $\quad$ $Q_2$ = 310 kN

$Q_3$ = 490 kN $\quad$ $Q_4$ = 390 kN $\quad$ $Q_5$ = 450 kN

25% of heavy vehicles (HV)

The second required parameter is the distance. Indeed, a distance has to be define between each vehicle generated. To do so, the PhD thesis of Claudio Baptista (Baptista, 2016) [10] has been taken as an inspiration. It has been generated randomly a uniform probability and then, using the inverse of a CDF curve, it has been determined a distance between each vehicle. Its parameters are taken from the PhD thesis of Claudio Baptista [10], as follow:

$$f(p) = gaminv(p, \alpha, \beta) \tag{8}$$

$d_0 = 120\ m$ → mean value

$d_m = 30\ m$ → modal value

$$\alpha = \frac{d_0}{d_0 - d_m} = 1.33 \tag{9}$$

$$\beta = d_0 - d_m = 90\ m$$

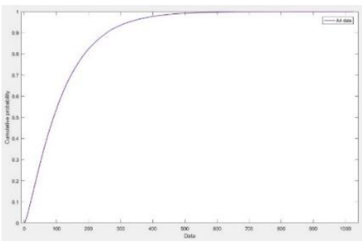

*Figure 11 : CDF curve used (from the software MatLab)*

Now that the traffic composition is define and also the distance between each vehicle, a traffic can be generated for one day. The number of vehicle for one day is 32'000 vehicles with 8'000 heavy vehicles, based on $N_{obs}$ = 2 x $10^6$ HV/year/lane for one traffic lane. However, it cannot represent the same as one-year traffic data. Thus, because of some IT performances, it has been generated one-week traffic data, which represents 160'000 vehicles (for five working days) with 40'000 HV, and it has been compared with those of one-day data.

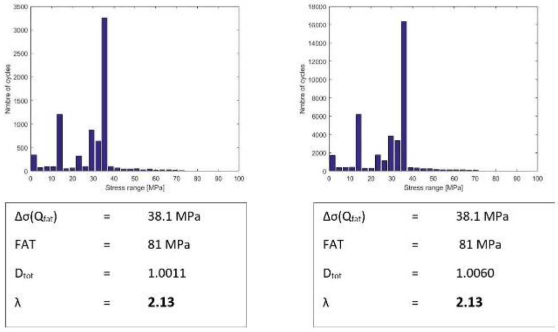


*Figure 12 : One-day data vs one-week data for stay C5*

The first observations show that the histograms and the obtained values are similar for the studied stays. It is true that there are some little differences but the peaks that characterise these histograms are presents for the same stress ranges. Thus, using one-day traffic data to generalize calculations may be considered as reasonable.

It is possible to determine the new damage equivalent factors for some selected stays, which are in common to have a midspan section.

| N° | $L_{crit}$ [m] | $\lambda_{3,5}$ | $\lambda_{4,6}$ |
|---|---|---|---|
| L12 | 54 | 2.32 | 2.00 |
| L14 | 54 | 2.38 | 2.05 |
| L7 | 71 | 2.63 | 2.25 |
| L9 | 71 | 2.38 | 2.02 |
| L3 | 81 | 2.24 | 1.94 |
| L1 | 89 | 2.25 | 1.95 |
| C1 | 89 | 2.34 | 2.03 |
| C5 | 105 | 2.44 | 2.13 |
| C9 | 129 | 2.27 | 2.12 |
| C13 | 150 | 2.37 | 2.13 |
| λ (Eurocode) | | 2.26 | 2.16 |
| λ (Maddah) | | 2.45 | 2.34 |
| Average | | 2.33 | 2.04 |

*Table 3 : λ factors for m=3,5 and m=4.6*

First of all, the obtained results have to be compare with the Eurocodes, based on a heavy traffic of 5 x $10^5$ HV/year/lane. The results must be multiplied by $\lambda_2 = 1.2233$. We do not take into account the partial factors $\lambda_3$ and $\lambda_4$ because they are equals to *1.0*. Keeping the hypothesis that λ1 is constant for critical lengths higher than 80m, it is possible to determine a value of the Eurocodes for a midspan section for critical lengths higher than 80m. This one is equal to *2.26*.

Then, for determining the value according to the researches of Nariman Maddah, the Figure 10 can be used for midspan section. The average value for the damage equivalent factor is equal to about *2.0*. As the PhD thesis of Nariman Maddah [9] is based on a heavy traffic of 5 x $10^5$ HV/year/lane, the obtained value is *2.45*.

Then, as described previously, stays are considered as tension components and thus, the S-N curve used is based on the slope's coefficients equal to 4 and 6 without cut-off limit. But for the Eurocodes and Maddah's thesis, the S-N curve used is based on the coefficients equal to 3 and 5 with a cut-off limit.

This difference, which is mainly occurred in the damage accumulation, allows to explain these little differences between the values. However, a question could be asked: "If there is a difference in the calculations of the λ factor considering the fatigue curves and the slope's coefficients, are formulas and values still valid for the fatigue verifications for stays?"

Indeed, the partial factors *λ1* and *λ2* are based on tests made with elements using fatigue curves for steel members, as described in the Figure 3. The EN 1993-2 [2] give a formula for calculating *λ2*, taking into account a *m* coefficient equal to 5. Trying to adjust this relation for stays, we can use a *m* coefficient of 6 to obtain:

$$\lambda_2 = \frac{Q_{m1}}{Q_0}\left(\frac{N_{obs}}{N_0}\right)^{1/6} = 1.1681 \qquad (10)$$

Moreover, the value of the lateral stay L7 is much higher than the others due to the approximation of its influence line has been bad made. Figure 13 show the comparison between the real influence line (in red) and the approximation (in blue). On can notice that even if the general shape is kept, the maximal peak value and the minimal one are not reached. For this reason, the lateral stay L7 is not taken into account for calculations of the average in the Table 3.

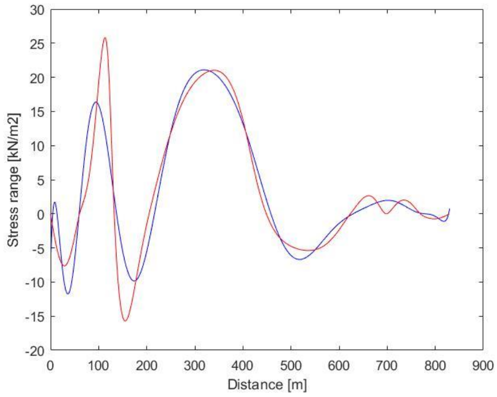


*Figure 13 : Approximation of the lateral stay L7*

In conclusion, one can notice that the λ factor values are slightly lower than those of the Eurocodes and Maddah's thesis for the calculations with slope's coefficients of 4 and 6 and slightly higher than the Eurocodes, as the results of Maddah's thesis, for those with coefficients of 3 and 5. For illustrating the trend of the results and to better compare them with the Eurocodes, the Figure 14 has been plotted.

One can notice clearly that the λ factors have a constant trend for critical length which vary from 54m to 150m for influence lines with a midspan section shape. This is not only to confirm hypotheses calculations of this project, which support the results of the Maddah's thesis, but also to ask questions about the value of the damage equivalent factor described in the Eurocodes.

Indeed, taking into account of the results of the Maddah's thesis, it can be observed that the constant trend is visible for the critical lengths higher than 80m but also for the one lower than 80m, while the Eurocodes telling us to take a decreasing value.

Hence, if the damage equivalent factor has a constant value for lengths varying from 54m to 150m, one can raise the question about the relevance of the critical length in the λ factor definition.

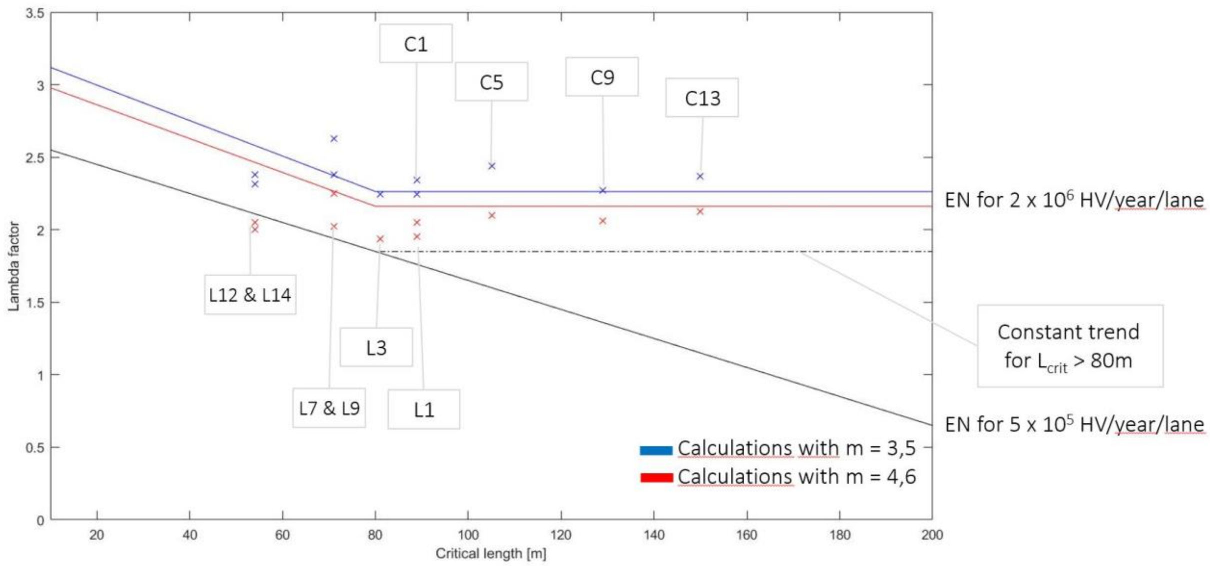


*Figure 14 : Comparison λ factors with m=3,5 and m=4,6*

## 8. Conclusion

One of the most important goals of this Master thesis was to verify if the damage equivalent factor method could be used to structural systems such as cable-stayed bridges. The first objective was to determine the stress influence lines in order to take into account the both internal forces acting in the composite deck, namely the flexion and the compression. The second objective was to calculate the damage equivalent factor, noted λ, for critical length higher than the Eurocodes limit of 80m.

The flexion and the compression have different behaviour inside the structure and hence create different influence lines. However, these maximal and minimal efforts are not necessarily at the same location that means we have to define which one is the most decisive. To do so, the idea is to determine influence lines based on the total stresses calculated with the sum of the two internal forces stresses, as described in the eq. (6). Moreover, Figure 7 shows that the stress based on the bending moment has more influence in the composite deck than the axial stress. In this fact, it is better to base the calculations on the maximal variation of the moments and add the associated variation of the axial forces to get the influence line which will better define the extreme stresses locations.

Researches done during this Master thesis showed that damage equivalent factors for midspan section remain constant when the critical length increase, apart from slight variations. This observation may be considered as a support for the results of the PhD thesis of Nariman Maddah [9], as described in the Figure 10. But it was more surprising to see that this trend works also for lengths lower than the Eurocodes limit, although it is described in the Eurocodes that the λ factor linearly decreases when critical length increases. Results of this project and those of the Maddah's thesis suggest that the damage equivalent factor remains constant for lengths varying from 50m to 150m. Thus, this would allow to simplify fatigue verification procedures if it is not necessary to define the critical length.

Taking into account long spans, one solution for the adjustment of the existing rules could be the next one. First, it would be better to define again the partial factor $\lambda_1$ for lengths varying between 20m and 200-300m. Then, it would be useful to define a new partial factor, noted $\lambda_5$, which would allow for taking into account the type of the fatigue curve.

Indeed, knowing that the Eurocodes define several S-N curves with different slope's coefficients, it would be effective to have a factor taking into account these coefficients.

## 9. Notation

C Constant representing the influence of the construction detail in fatigue strength expression

D, d Damage sum, damage

M Bending moment in Nm

N Axial effort in N ; Number of cycles

$b_{eff}$ Effective width of the concrete slab in m

m Fatigue curve slope coefficient

n Number

$\Delta\sigma$ Stress range

$\Delta\sigma_C$ Fatigue strength under direct stress range at 2 million cycles in N/mm$^2$

$\Delta\sigma_D$ Constant amplitude fatigue limit (CAFL) under direct stress range at 5 million cycles in N/mm$^2$

$\Delta\sigma_L$ Cut-off limit under direct stress range at 100 million cycles in N/mm$^2$

$\Delta\sigma E2$ Equivalent direct stress range compute at 2 million cycles in N/mm$^2$

$\gamma_{Ff}$ Partial safety factor for fatigue action effects

$\gamma_{Mf}$ Partial safety factor for fatigue strength

$\lambda$ Damage equivalent factor

$\lambda_1$ Factor accounting for span length (in relation with the length of the influence line)

$\lambda_2$ Factor accounting for a different traffic volume than given

$\lambda_3$ Factor accounting for a different design working life of the structure than given

$\lambda_4$ Factor accounting for the influence of more than one load on the structural member

$\lambda_{max}$ Maximum damage equivalent factor value, taking into account the fatigue limit

## 10. References


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