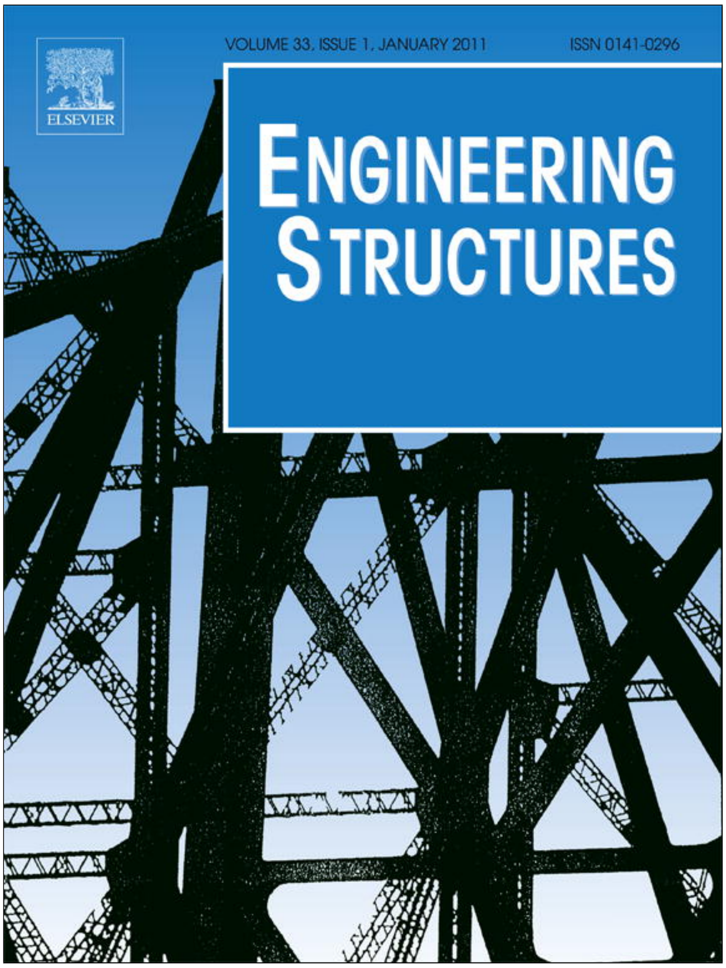
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Fatigue assessment of highway steel bridges in presence of seismic loading

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ABSTRACT

In this study a LEFM (Linear Elastic Fracture Mechanics) approach is used in a probabilistic context to evaluate the fatigue reliability of steel girder highway bridges in the presence of seismic loading. In the first part the fatigue damage is related to the traffic load produced by heavy trucks crossing the bridge; the second part deals with the fatigue damage related to seismic loading. Both damage typologies are analyzed using linear elastic fracture mechanics principles, and the time required for an initial crack propagation is calculated. Taking into account that the correlation between fatigue effects and seismic actions is not usually considered in the literature, this method could enable a better understanding of progressive damage phenomena due to fatigue related problems, and could give some new insights for increasing the remaining fatigue life of a large number of steel bridges in seismic zones.

1. Introduction

The ASCE Committee on Fatigue and Fracture Reliability [1] reported that 80%–90% of failures in steel structures are related to fatigue and fracture, and this data is confirmed by Byers et al. [2]. Steel girder bridges are very common and are expected to be vulnerable to fatigue and fracture-related problems, as mentioned by Raju et al. [3]. The problem of fatigue assessment [4–7] becomes further complicated if the deteriorated conditions of existing steel need to be considered: defects of superstructures represent the 20.5% of the causes for the replacement of steel highway bridges [8], while each year about 1200 bridges reach the end of their design life [9]. Most of them must be strengthened, repaired or rebuilt to ensure an acceptable level of safety considering present and future traffic conditions. Flaws are expected to be present in steel structures, in terms of defects in welds, notches, dents, etc. Cracks originating from these inherent 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 to a critical size, the structure fails [10]. These effects could be more significant when a seismic event strikes the structure during its service life. In this context, to authors' best knowledge a correlation between fatigue effects and seismic actions has not been extensively analyzed in the literature: this is a crucial argument in bridge engineering, especially to estimate the precise cumulative effect of the total damage and assess 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 [11,12]. Moreover, interesting considerations on safety assessment under multi-hazards can be found in [13].

In this paper, a method for fatigue damage estimation of highway steel bridges in presence of seismic loading is presented. 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.

A rational procedure for the examination of fatigue safety which proceeds by step levels using both deterministic and probabilistic methods is appropriate in most cases. Probabilistic methods enable the explicit consideration of the scatter of the parameters that influence the fatigue strength and the fatigue damaging effect.

The main objective of this paper is to introduce an innovative method that allows examination of the fatigue safety and determination of the remaining fatigue life based on a step-by-step procedure referring to LEFM (Linear Elastic Fracture Mechanics). The first part of the work deals with the fatigue analysis using crack propagation law based on LEFM, coupled with a reliability method involving heavy traffic loads; the second part deals with the fatigue damage caused by seismic loading and analyzed with the same LEFM theory. Results are finally discussed in relation to a case study.

The first assumption of this work is that fatigue and seismic hazards are not considered to interact in probabilistic terms, as

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for e.g. reported in other approaches [13]. The present study is focused on damage estimation due to the seismic load, studied in deterministic terms, and traffic induced fatigue damage taken into account in probabilistic terms. The second assumption is related to the target reliability index which has been treated as constant in time according to ISO2394, ISO13822, SAMCO [14] and Sustainable Bridge [15]. 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 seismic damage as coupled processes in probabilistic terms.

2. Fatigue due to traffic loading

In the first part of this study, the fatigue analysis based on LEFM, coupled with a reliability method involving heavy traffic loads is developed. The procedure is then applied to a particular case study represented by a single span steel girder highway bridge.

2.1. Crack propagation law

Due to the inherent disadvantage of the S–N curve approach, which cannot incorporate information on crack size, an alternative approach based on LEFM concepts [16,17] is considered in this study. The LEFM approach is based on crack propagation theory [18–21]. Paris' law, the most common LEFM-based crack growth model, is used since it retains the simplicity of the fatigue evaluation process. This can be described as:

$$\frac{da}{dN} = C \cdot \Delta K^m \tag{1}$$

where a is the crack size, N is the number of stress cycles, C and m are material constants and ΔK is the stress intensity range. According to LEFM theory [17], ΔK can be estimated as:

$$\Delta K = F(a, Y) \cdot \Delta\sigma \cdot \sqrt{\pi \cdot a} \tag{2}$$

where $\Delta\sigma$ is the tensile stress range, $F(a, Y)$ is the geometry function to take into account possible stress concentrations and Y is a vector of geometrical parameters [22], such as the stress concentration coefficient and the dimensions of the specimen under consideration. The geometry function is expressed as the product of four separate factors [23,17]:

$$F(a, Y) = F_g \cdot F_w \cdot F_s \cdot F_e \tag{3}$$

where F_e, F_s, F_w and F_g are crack shape, free surface, finite width and stress gradient correction factors, respectively [23,17]:

$$F_e = \frac{1}{\int_0^{\frac{\pi}{2}} \sqrt{1 - \frac{c(a)^2 - a^2}{c(a)^2} \sin^2(\vartheta)}\, d\vartheta} \tag{4}$$

$$F_s = 1.211 - 0.186\sqrt{\frac{a}{c(a)}} \tag{5}$$

$$F_w = \sqrt{\sec\frac{\pi a}{2t_f}} \tag{6}$$

$$F_g = \frac{-3.539 \ln\frac{z}{t_f} + 1.981 \ln\frac{t_{cp}}{t_f} + 5.798}{1 + 6.789\left(\frac{a}{t_f}\right)^{0.4348}}. \tag{7}$$

In the previous expressions z is the weld leg size, t_f is the flange thickness, t_{cp} is the cover plate thickness, a is the crack depth, c is half the crack length as a function of crack depth, and ϑ is the angle for an elliptical crack. The relation between c and a is given by $c(a) = 3.549a^{1.133}$ [23,17].

Hence the crack propagation law can be written as:

$$\frac{da}{dN} = C[F(a, Y) \cdot \Delta\sigma \cdot \sqrt{\pi \cdot a}]^m. \tag{8}$$

The integration of Eq. (8) is quite difficult, but it can be solved according to the Kunz method [16], where $F(a, Y)$ assumed to be constant and equal to Y. The crack propagation law becomes:

$$\frac{da}{dN} = B \cdot \Delta\sigma^m \cdot a^{m/2} \tag{9}$$

in which $B = C \cdot Y^m \cdot \pi^{m/2}$.
Eq. (9) can be written as:

$$\int_{a_0}^{a_{crit}} a^{-m/2} da = \int_0^{N_f} B \cdot \Delta\sigma^m dN \tag{10}$$

in which a_{crit} is the critical depth of the crack at a number of cycles equal to N_f (f standing for "failure"). Fatigue failure is reached at a number of cycles, where a crack depth of half plate thickness has been reached [24]. The initial size of the crack a_0 is assumed as varying from $a_0 = 0.075$ mm to $a_0 = 0.4$ mm [18,24]. These may be considered as lower and upper bound values, respectively, for the initial crack size in the welded plate test specimen [25,26]. Based on the size of the defects in actual welded joints and on values reported in the literature [27,28], in the present investigation the initial crack depth is assumed as $a_0 = 0.1$ mm. Considering the relationship between stress and cycles in the S–N curves: $\Delta\sigma_c^m \cdot N_c = const.$ and integrating Eq. (10), we obtain:

$$\frac{2}{2-m}\left(a_{crit}^{(2-m)/2} - a_0^{(2-m)/2}\right) = B \cdot \Delta\sigma_c^m \cdot N_c \tag{11}$$

in which $\Delta\sigma_c$ represents the fatigue strength at $N_c = 2 \times 10^6$ cycles.

In Eq. (11) the term B can be evaluated as:

$$B = \frac{\frac{2}{2-m}\left(a_{crit}^{\frac{2-m}{2}} - a_0^{\frac{2-m}{2}}\right)}{\Delta\sigma_c^m \cdot N_c} = C \cdot F(a, Y)^m \cdot \pi^{m/2} \tag{12}$$

and

$$C = \frac{\frac{2}{2-m}\left(a_{crit}^{\frac{2-m}{2}} - a_0^{\frac{2-m}{2}}\right)}{\Delta\sigma_c^m \cdot N_c} \cdot \frac{1}{F(a, Y)^m \cdot \pi^{m/2}} = \frac{B}{F(a, Y)^m \cdot \pi^{m/2}}. \tag{13}$$

The crack growth exponent m is a function of the material: for structural steels it is commonly assumed as 3.0. Using the finite difference method, it follows that:

$$a_{i+1} = C\left[\Delta\sigma_{eq}\sqrt{\pi a_i} F(a_i, Y)\right]^m (N_{i+1} - N_i) + a_i \tag{14}$$

in which the stress parameter involved is the equivalent constant amplitude stress range $\Delta\sigma_{eq}$ defined as:

$$\Delta\sigma_{eq} = \left[\frac{\sum_i (n_i \cdot \Delta\sigma_i^m)}{N}\right]^{1/m} \tag{15}$$

in which n_i is the number of cycles of stress range $\Delta\sigma_i$; $\Delta\sigma_i$ is the variable amplitude stress range; N is the total number of cycles; m is the slope of the corresponding constant S–N line. Crack propagation is related with stress cycles caused by heavy vehicles crossing the bridge. These stress cycles produce fatigue damage in terms of crack propagation.

2.2. Damage accumulation

It is not completely clear how the stress cycles below a constant amplitude fatigue limit affect the fatigue life [25]. Stress cycles due to trucks are usually lower than the fatigue limit [3] hence, according to Miner's rule [29], they should not produce any damage even if this approach, in some situations, could not stand on the safe side. The damage model developed in this paper considers the damage due to stress cycles below the cut-off limit which causes damage in terms of crack propagation according to LEFM principles [30]. The adopted damage model implies that stress ranges are damage effective only if $\Delta\sigma_{th}$ is exceeded, where $\Delta\sigma_{th}$ is the cut-off limit and $\Delta\sigma_D$ is the fatigue limit for constant amplitude stress ranges at the number of cycles $N = 5 \times 10^6$, defined by [16]:

$$\Delta\sigma_{th} = \Delta\sigma_D \cdot f(D) = \Delta\sigma_D \cdot \frac{F(a_0, Y)\sqrt{\pi a_0}}{F(a, Y)\sqrt{\pi a}}. \quad (16)$$

This is also called damage limit, and is no longer constant but decreases with increasing crack size and with the increments of damage. A single damage increment, taking $\Delta\sigma_{th}$ as the cut-off limit and $\Delta\sigma_k$ as the category detail at the number of cycles $N = 2 \times 10^6$, according to Kunz [16] is represented by:

$$d_i = \frac{\Delta\sigma_i^m - \Delta\sigma_{th}^m}{\Delta\sigma_k^m - \Delta\sigma_{th}^m} \cdot \frac{1}{N_k} \quad (17)$$

where:

$\Delta\sigma_i$ = applied stress range

m = S–N curve slope

D = total damage.

Failure will occur when the accumulated damage $D = \sum d_i = 1$, according to Miner [29].

2.3. Reliability method

The probability of crack detection during inspection and monitoring operation is generally evaluated at an intermediate stage, and subsequently linked to the calculated probability of fatigue fracture to obtain the probability of failure [15]:

$$p_{fail} = p_{fat}(1 - p_{det}) \quad (18)$$

where:

p_{fail} = probability of failure

p_{fat} = probability of fatigue fracture

p_{det} = probability of detection.

The probability of failure can also be expressed with the reliability index according to the standard normal distribution. The reliability of a structural element is compared to the target value:

$$\beta_{fail} \geq \beta_{target} \quad (19)$$

where:

β_{fail} = reliability index with respect to failure

β_{target} = target reliability index.

This model adopts the fatigue action effect (the required nominal fatigue strength) as a so called required operational load factor (α_{req}), which is obtained by dividing the required nominal fatigue strength by the action effect of the fatigue load model applied [16]:

$$\alpha_{req} = \frac{\Delta\sigma_{C,req}}{\Delta\sigma(\Phi Q_{fat})}, \quad (20)$$

where:

$\Delta\sigma_{C,req}$ = required nominal fatigue strength

α_{req} = required operational load factor

$\Delta\sigma(\Phi Q_{fat})$ = stress range due to the load model adopted at worst position (e.g. maximizing the fatigue stress amplitude).

For a simplified probabilistic approach, a relation between mean value of required operational load factor $m(\log\alpha_{req})$ and number of future truck passages N_{fut}, established by Kunz [16], could be used. This method has been improved in this study: the mean of the required operational load factor $m(\log\alpha_{req})$ is read for the chosen fatigue category, starting from a predefined number of future trains N_{fut} (as from 2008), derived from the aforementioned analysis of real traffic. The relation is useful for every influence length and whatever commissioning time. According to the same model, the value of 0.04 may be taken as the standard deviation of the required operational load factors, resulting from the assumed fuzziness of the traffic model [15]. Adopting the following notation and assumption: $s_E = s(\log\alpha_{req}) = 0.04$

$$\beta_{fat}(N_{fut}) = \frac{m_r + m_E(N_{fut})}{s_r^2 + s_E^2} \quad (21)$$

where:

s_E = standard deviation of the required fatigue strength

$\beta_{fat}(N_{fut})$ = reliability index

$m_R = \log\Delta\sigma_c + 2s_R$ = mean of the fatigue strength relating to $N = 2 \times 10^6$

$m_E(N_{fut}) = m(\log\alpha_{req}) + \log\Delta\sigma(\Phi Q_{fat})$ = mean of the required fatigue strength as a function of the number of future trucks N_{fut}, number of truck cycles

$s_R = \frac{(s\log N)}{m}$ = standard deviation of the fatigue strength (MPa)

m = slope of the S–N curve

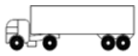
$s(\log N)$ = standard deviation of test results.

According to this model, $\beta_{fail} = \beta$, and the β value obtained should be compared to the target reliability index. Concerning the choice of target reliability index, it has been considered that for Serviceability Limit States specific values of β are recommended for a given remaining service life, according [31] Basis for Design of Structures. For the fatigue limit state in the assessment of existing structures and a remaining service life of 50 years, a value of $\beta = 2.3$ is recommended in case of inspection and $\beta = 3.1$ if the element or detail is not visible [31].

The reliability model herein presented has been based on Kunz [16] hypothesis, but has been improved by adopting actual traffic analysis and applied for the case study aforementioned.

The method, shown in the previous paragraph, is applied to a steel girder highway bridge with a common scheme. The bridge is supposed to be have been built in 1960 and is on the most important highway in Italy (i.e. the Milan–Rome–Naples (A1)). Heavy vehicle traffic data has been collected from AISCAT data [32, 33] and is shown in Fig. 1 in terms of daily number of heavy vehicles crossing the bridge. From this the annual number of trucks has been calculated. The present study considers two different load models for heavy traffic, Model A and Model B: Model A consists in heavy traffic loads provided by Codes (see Table 1) from 1960 to 2008 in terms of weight/axle and the corresponding distribution of weight/axle for all truck's types, and is shown in Table 1; Model B takes into account the composition of heavy traffic as reported by this specific highway, distinguishing three types of

Table 1
Load Model A, truck typologies.

Code	Period	Truck	Total weight (kN)	Number of axes	Distances (m)	Weight axes (kN)	Traffic composition (%)
Ministero I. e T. [34]	1930–1961		120	2	3.00	40 80	100
Ministero I. e T. [35]	1962–1979		120	2	3.00	40 80	100
Ministero I. e T. [36]	1980–1989		310	3	4.00 1.50	70 120 120	100
Ministero I. e T. [37]	1990–2007		300	3	4.00 1.50	100 100 100	100
Ministero I. e T. [38]	2008–future		200	2	4.50	70 130	20
			310	3	4.20 1.30	70 120 120	5
			490	5	3.20 5.20 1.30 1.30	70 150 90 90 90	50
			390	4	3.40 6.00 1.80	70 140 90 90	15
			450	5	4.80 3.60 4.40 1.30	70 130 90 80 80	10

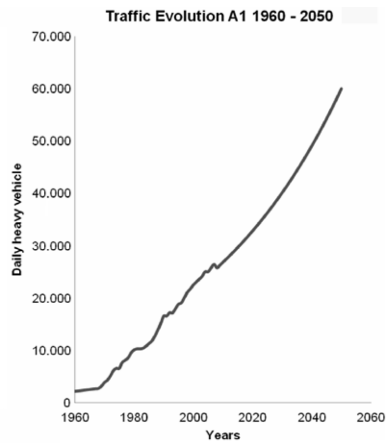


Fig. 1. Heavy vehicle evolution year by year in Milan–Rome–Naples highway (A1).

truck unchanged over years, and the distribution of weight/axle as provided by the Highway Code [32,33]; it assumes also that 50% of trucks run with a full load; the distribution of weight/axle is shown in Table 2. Moreover, while Model A provides loads differentiated by time period, according to different codes during the life of the bridge, Model B provides a system of loads which is considered to be the same from past to future times. Both load models consider trucks crossing the bridge along its slow traffic line and the resulting stress ranges calculated for both models are shown in Tables 3–4.

The fatigue damage is calculated in terms of bending of the main girder at the half span section; the detail which has been examined is the cover plate of the bottom flange of the main girder, characterized by Category 140 for bending loads. Fig. 2 shows the transverse section of the girder, and Fig. 3 the typical fatigue failure. The reliability analysis of details has been performed assuming the methodology described above and adopting the aforementioned load models. In Fig. 4 is reported the analysis from 2010 to future for load Model A. The same analysis related to load Model B is shown in Fig. 5. As shown in the previous figures for the investigated detail, Model A (35 years of remaining life) could be considered safer than Model B (40 years of remaining life). Moreover the fatigue assessment has been carried out by adopting Miner's damage model [29]. This cumulative damage approach implies the use of the formula:

$$D_{d,EC_a} = \sum_{i}^{n} \frac{n_{E_i}}{N_{R_i}} \leq 1.0,$$

where n_{E_i} is the number of cycles associated with the stress range $\gamma_{F_f} \Delta\sigma_i$ for band "i" in the factored spectrum, (MPa); N_{R_i} is the endurance (in cycles) obtained from the factored $\frac{\Delta\sigma_C}{\gamma_{M_f}}$ vs. N_R curve for a stress range of $\gamma_{F_f} \Delta\sigma_i$ (MPa), $\Delta\sigma_C$—reference value of the fatigue strength at $N_C = 2$ million cycles (MPa); γ_{M_f}—partial factor for fatigue strength $\Delta\sigma_C$. The constant amplitude damage was calculated by adopting the two-slope S–N curve of Instruction 44/F (1992).

Miner's damage model has been considered to achieve a comparison with the actual Italian code which adopts this approach [38]: this implies that the traffic spectra reported in Table 1 from 2008 to future has to be implemented for the whole service-life period. This assessment leads to a total fatigue life of 68 years, implying that the structure has zero remaining life: this could be explained since the model is related to unreal traffic

Table 2
Load Model B, truck typologies.

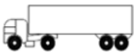
Trucks	Number of axes	Weight modality	Total weight (kN)	Weight axes (kN)	Distances (m)	Traffic composition (%)
				100		
		50% full	300	100		
				100	4.00	
	3				1.50	15
				50		
		50% empty	120	35		
				35		
				100		
				100		
		50% full	400	100		
				100	3.40	
	4				6.00	10
				60	1.80	
				30		
		50% empty	150	30		
				30		
				60		
				80		
		50% full	440	100		
				100		
				100	3.20	
	5				5.20	75
				60	1.30	
				25	1.30	
		50% empty	160	25		
				25		
				25		

Table 3
Load Model A, stress ranges.

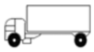
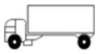
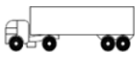
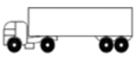
Code	Period	Truck	$\Delta\sigma$ (MPa)	$\Delta\sigma_{eq}$ (MPa)
Ministero I. e T. [34]	1930–1961		16.30	16.30
Ministero I. e T. [35]	1962–1979		16.30	16.30
Ministero I. e T. [36]	1980–1989		41.09	41.09
Ministero I. e T. [37]	1990–2007		40.62	40.62
			26.50	
			41.17	
Ministero I. e T. [38]	2008–future		61.57	53.54
			46.54	
			52.43	

Table 4
Load Model B, stress ranges.

Trucks	Weight modality	$\Delta\sigma$ (MPa)	$\Delta\sigma_{eq}$ (MPa)
	Full Empty	44.78 17.7	
	Full Empty	53.5 19.14	47.90
	Full Empty	62.4 21.3	

estimation, cumulating damage related to actual loadings also for the past.

3. Effect of seismic loading on fatigue damage

The fatigue strength of structures could be considerably reduced due to large cyclic loadings imposed by earthquakes [39]. In this context, the crack due to a seismic event has been evaluated to clarify the effect of seismic loading on the fatigue strength of welded joints. The behaviour of the structural steel is assumed as isotropic and linear elastic during the seismic event, hence the damage accumulation due to seismic loading can be described by LEFM analysis.

The main observation is that small cracks arising and propagating due to seismic loads enhances damage accumulation for subsequent variable amplitude loading due to traffic loads.

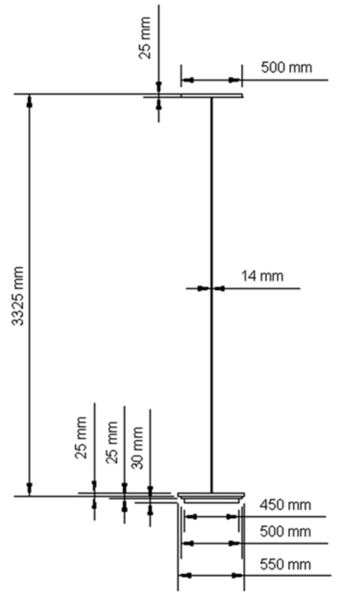


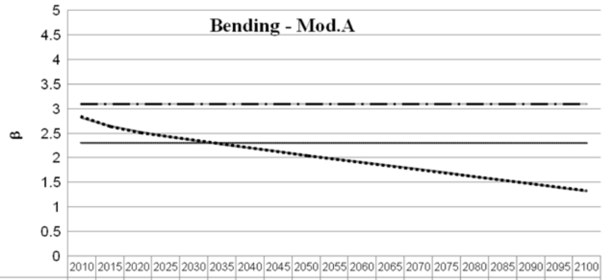
Fig. 2. Transverse section of the girder.

The crack propagation law [40] can be written using the finite difference method as follows:

$$a_{seismic} = C \cdot \left[\Delta\sigma_{seismic} \cdot F\left(a_{traffic}, Y\right) \cdot \sqrt{\pi a_{traffic}}\right]^{m} \times \left(N_{seismic}\right) + a_{traffic} \qquad (22)$$

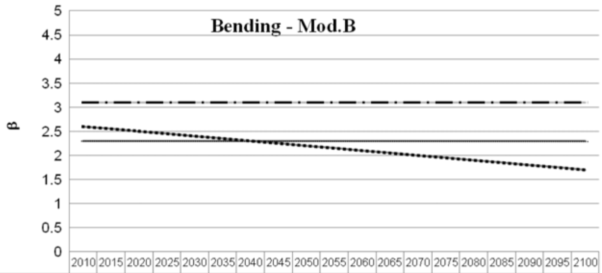
in which the seismic crack size, $a_{seismic}$, is a function of stress cycles due to earthquakes, $N_{seismic}$, and depends on crack propagation due to traffic loads accumulated prior to the seismic event. A peculiar characteristic of seismic loading is the application of a few cycles of highly variable amplitude stress; Yamada [25] and Carpinteri et al. [41] highlighted that coefficients C and m in Paris' law depends on R value, which represents the ratio between minimum and maximum stress in a stress spectrum. Yamada [25] provides values of C and m for different R values for variable amplitude loadings. In this study C and m values have been evaluated by linear regression from those provided by Yamada [25] depending on R values for the seismic loading (see Figs. 6 and 7).

The same bridge studied in the first part of this work is now analyzed including the effect of the seismic loading. A PGA (Peak Ground Acceleration) of 0.35g is considered. A simplified FEM model of the structure has been realized, implementing horizontal and vertical spectrum compatible accelerograms. Then a dynamic analysis has been carried out, and time dependent stress spectra have been evaluated at the bottom flange of the main girder at mid-span, see Fig. 8. The total number of cycles for each spectrum and maximum and minimum stress values of each cycle have been recorded by means of a routine implemented for this purpose: the output file is shown in Fig. 9. The frequency distribution of stress ranges is shown in Fig. 10. Finally the equivalent stress has been calculated using Eq. (15) and crack propagation due to seismic loading has been evaluated using Eq. (22). Crack effects due to seismic action have been coupled with the traffic crack propagation to evaluate the coupled damage in terms of reliability against fatigue due to traffic and seismic loading during the service life of the bridge. The crack propagation due to traffic and seismic loading vs. years of service is shown in Fig. 11. It was assumed that an earthquake will occur in year 2050. A sensitivity analysis varying the year of the seismic event occurring has been developed, see Fig. 12; it can be observed that an anticipated earthquake can lead the structure to an anticipated failure with respect to the earthquake with the same intensity in future times.



	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060	2065	2070	2075	2080	2085	2090	2095	2100
Beta_min	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
Beta_max	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
Beta	2.82	2.65	2.53	2.44	2.35	2.27	2.20	2.13	2.05	1.98	1.91	1.84	1.77	1.70	1.62	1.55	1.47	1.39	1.32

Fig. 4. Reliability analysis for Model A.



	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060	2065	2070	2075	2080	2085	2090	2095	2100
Beta_min	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
Beta_max	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
Beta	2.60	2.55	2.50	2.45	2.40	2.35	2.30	2.25	2.20	2.15	2.10	2.05	2.00	1.95	1.90	1.85	1.80	1.75	1.70

Fig. 5. Reliability analysis for Model B.

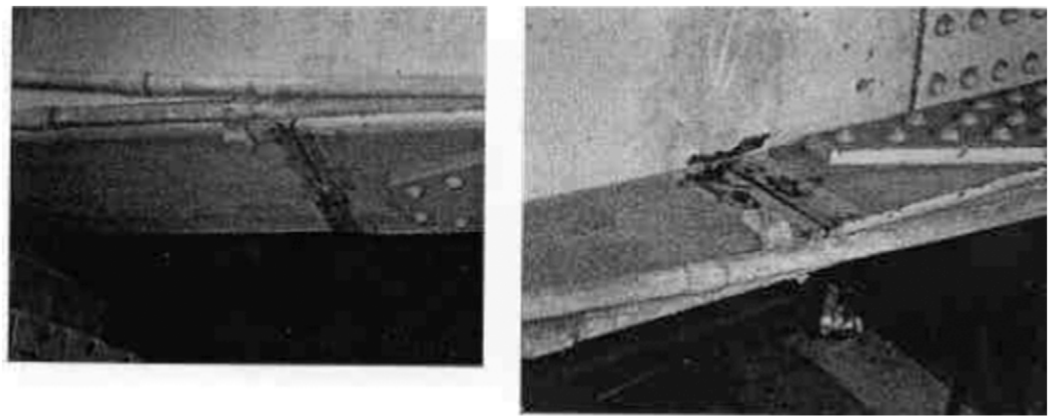


Fig. 3. Typical fatigue failure detail [15].

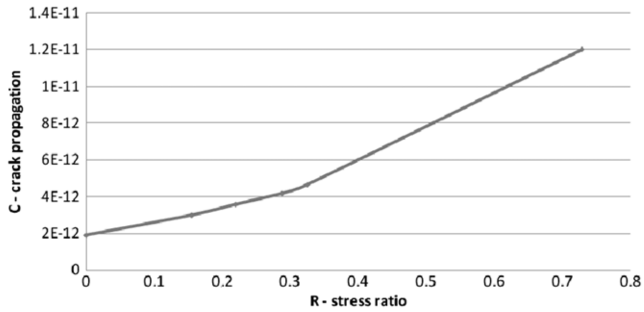


Fig. 6. Coefficient C depending on R ratio.

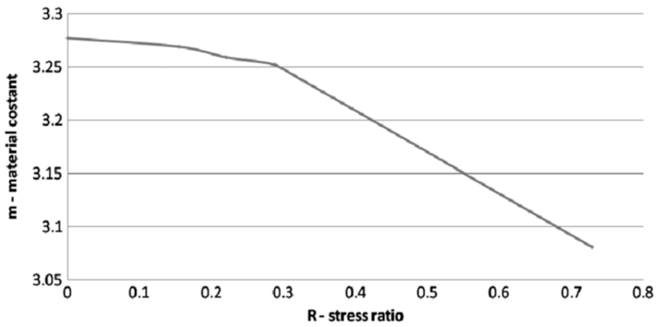


Fig. 7. Exponent m depending on R ratio.

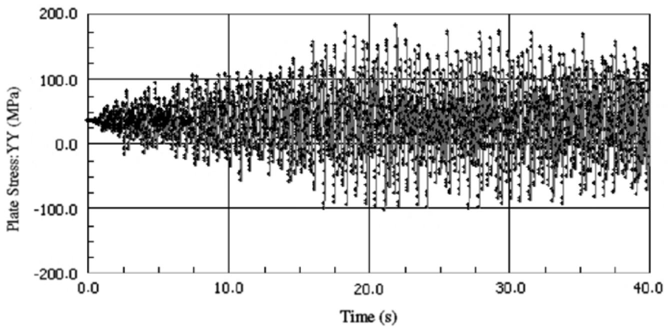


Fig. 8. Time dependent stress results.

The reliability analysis has been carried out considering this more damaging situation. As an example, an earthquake has been considered striking the structure during year 2015; Fig. 13 shows the difference between the trend of the reliability index with and without the seismic event. The "out of service" situation is reached in 2020 instead of 2035.

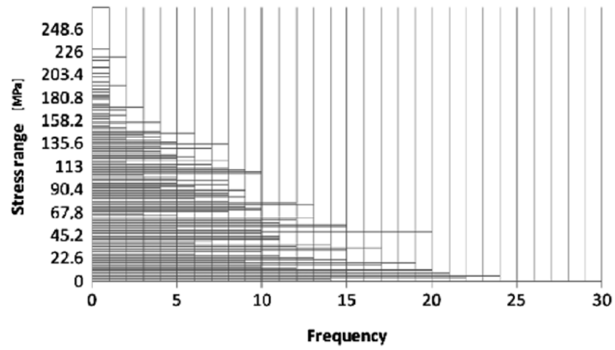


Fig. 10. Frequency distribution.

4. Conclusions

A reliability-based LEFM approach considering flaws in welding is proposed to evaluate fatigue damage in highway bridges due to traffic loading and supposing that at least one seismic event occurs during the service life of the structure. A typical steel girder highway bridge has been analyzed by reliability analysis, taking into account a wide variety of traffic and load models, either for the past and the future. Results have shown that the considered detail, assuming the load Model A, will reach "out of service" situation in 2035, whereas the same detail assuming the Model B terminates in 2040. Assessment based on Miner's rule may give conservative predictions of the fatigue life, whereas the LEFM method allows one to consider a more realistic damage accumulation.

Seismic loading contributes to increase the damage of the structure: in fact, structures that are not seriously damaged by an earthquake continue to be used after the seismic event but, due to this event, the residual fatigue life could be greatly reduced. As a matter of fact seismic action coupled with traffic can lead the bridge to an earlier failure than considering just traffic loading, since the occurrence and propagation of small cracks due to seismic loads increases damage accumulation. This study enables a better understanding of the significance of the various traffic spectra, and gives some new insights towards a more reliable prediction of the remaining life of bridges taking into account the effects of seismic action combined with traffic. In this context, infrastructure agencies should explore this key issue, being aware that the formation of small cracks due to traffic loading could be amplified by seismic loading enhancing subsequent fatigue damage accumulation. As a result, it is crucial to keep all stress variations due to service loading after an earthquake far below the fatigue limit of pre-cracked joints in order to avoid fatigue damage accumulation and unexpected collapse.

The approach can be considered as first step towards a more sophisticated analysis of the problem which will take into account the interaction between fatigue and seismic damage as coupled processes in probabilistic terms.

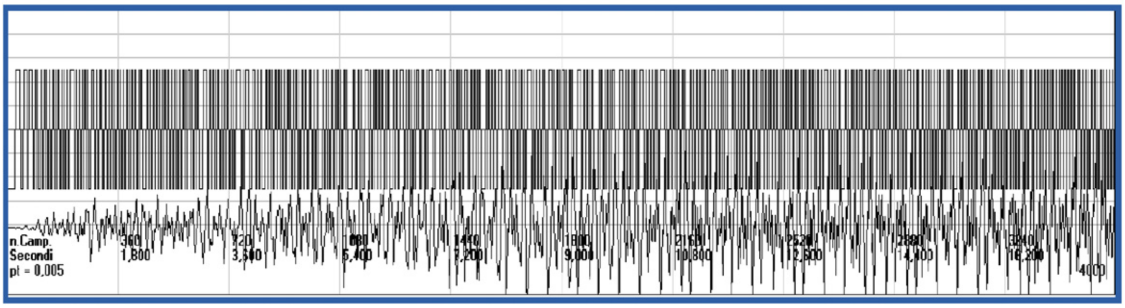


Fig. 9. Output file representing time dependent stress results.

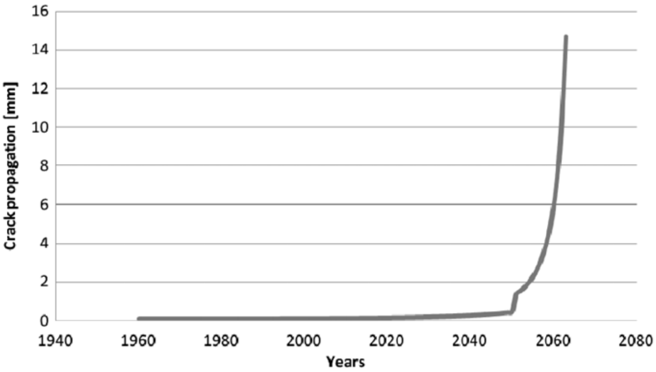


Fig. 11. Crack propagation vs. year of service due to traffic and seismic loading.

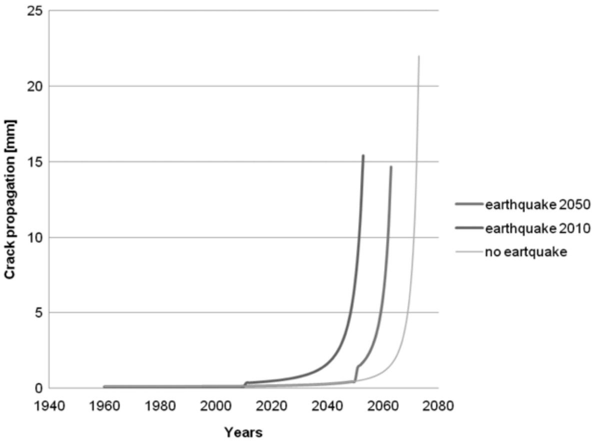
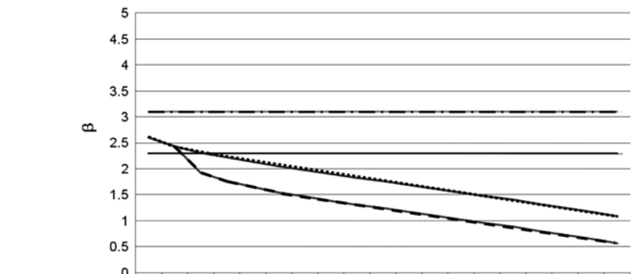


Fig. 12. Sensitivity analysis varying the year of the seismic event.



	2010	2015	2020	2025	2030	2035	2040	2045	2050	2055	2060	2065	2070	2075	2080	2085	2090	2095	2100
Beta_min	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3
Beta_max	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1	3.1
Beta_eartquake	2.60	2.43	1.92	1.76	1.65	1.54	1.47	1.38	1.31	1.24	1.17	1.10	1.02	0.95	0.88	0.80	0.73	0.65	0.57
beta_no earthquake	2.60	2.43	2.31	2.22	2.13	2.05	1.98	1.90	1.83	1.76	1.69	1.62	1.54	1.47	1.40	1.32	1.25	1.17	1.09

Fig. 13. Reliability analysis, coupling traffic and earthquake damage, or traffic damage only.

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