



Traffic simulations with structural evaluation indexes computation Part 2: Fatigue

**Verkehrssimulationen mit Berechnung der strukturellen
Auswertungsindexe
Teil 2 : Ermüdung**

**Simulations de trafic intégrant la détermination d'indices de
performance structurale
Partie 2 : Fatigue**

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**Projet de recherche AGB 2010/003 sur demande du groupe de travail
recherche en matière de ponts (AGB)**

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Summary

The objective of this research project is to combine the two domains of traffic simulation and structural performance determination in order to better grasp heavy traffic effect on pavement and road bridges, both new and existing ones, by using adapted traffic simulations. The results of the research have been divided into two parts which join in a complete methodology allowing for the structural assessment of road bridges and pavements. This report, Part 2, presents the results of the research regarding the structural assessment of road bridges under traffic load, in particular against fatigue. The other report, Part 1, deals with traffic and the effect of it on pavement. In order to facilitate the reading, common general introduction and conclusions to both reports are included in each one of them.

This summary deals now with Part 2. Fatigue is the third cause for damage of steel structures, while it ranks as first cause for damage of steel bridges. In fatigue analysis of bridges, independently from the material, one uses traditionally the nominal stress approach, wherein constant amplitude (CA) characteristic S-N curves are used in combination with modified Miner's linear damage accumulation rule. CA characteristic S-N curves, which related the nominal applied stress range, S , to the p -quantile of fatigue life, N , are obtained from fatigue tests at constant stress amplitudes.

In current standards CA characteristic S-N curves are estimated by fitting a linear regression to the experimental failure data points but this approach has limitations which may substantially affect the accuracy of fatigue assessment of existing bridges as well as design of new ones. Several authors proposed to overcome the limitations related to the CA S-N curves by using Maximum Likelihood (ML) method to fit a random CAFL non-linear model to experimental data points (failures and run-outs). Nevertheless, this approach does not give explicit method to compute p -quantile S-N curve from ML estimate of model parameters. Furthermore, since the fitted model is non-linear, the direct comparison with current standards is not straightforward. Concerning the limitations related to use of modified Miner's rule, many research works were carried out to investigate on the damage effect of stress range cycles below the CAFL as well as on the choice of the critical value of damage sum. However, these works only provide general trends and do not present a rigorous statistical approach overcoming the shortcomings of the modified Miner's rule.

This research provides a rigorous probabilistic approach for estimation of: 1) Characteristic S-N curves for CA and VA fatigue loadings; and 2) Critical damage sum parameter, used in Miner's rule. The new probabilistic approach also includes a new framework for fatigue reliability assessment of existing bridges. The novel contributions of the presented approach consists in: 1) Using Monte-Carlo Simulations (MCS) method for determining true CA characteristic S-N curve, after having estimated CA S-N model parameters with ML method; 2) Using a new scheme which combines MCS and ML methods and uses VA experimental results to estimate the VA S-N curve reduced slope, m_2 , as well as the critical damage sum random parameter, D_c ; 3) Giving more insight into realistic bridge fatigue reliability indexes, by defining a new framework which computes the evolution of probability of fatigue failure event (i.e. remaining fatigue life) as joint probability of CAFL exceeding event and critical damage accumulation event. Finally, considerations on the fatigue checks formats from the Eurocodes and the partial safety factor values for fatigue were made, including a new framework for re-calibration of fatigue checks partial safety factors. Even if applied herein to welded joints, the developed tools have general applicability.

The application to three study cases of ML-MCS approach for estimation of CA S-N curves and the comparison with current standards, allowed for identifying inaccurate definition of fatigue strength in current standards, especially in high cycle fatigue (HCF) region. Moreover, the application to two study cases of ML-MCS approach for estimation

of m_2 and D_c parameters and the comparison with current standards, revealed an inadequate definition of damage accumulation mechanism in application of Miner's rule using current standards. The application of reliability analysis framework to the Venoge bridge study case showed conservatism but yet inaccuracy of Eurocode-based assessment of fatigue reliability index, during the 100 year-design life of the bridge. The revision of Eurocode formats for fatigue design of structures for two fatigue sensitive details, explained the inexactness of hypothesis of considering the same partial resistance factors for all verification formats. It further showed that Eurocode design format, also valid for existing bridge assessment, which is based on CAFL is not on the safe side. More thoughts differentiating existing from new bridges with respect to S-N curves and re-calibration of fatigue design partial safety factors should be made.

Keywords: fatigue reliability, S-N curves, existing bridges, steel bridges, Monte-Carlo Simulations, Maximum Likelihood, reliability index, partial safety factors.

Résumé

L'objectif de cette recherche est de combiner les deux domaines que sont la simulation du trafic et la détermination de la performance structurale afin de mieux appréhender l'effet du trafic lourd sur les chaussées et les ouvrages d'art routiers, neufs ou existants, ceci en recourant à des simulations de trafic adaptées. Les résultats de la recherche ont été séparés en deux parties qui se rejoignent dans une méthodologie complète permettant l'évaluation structurale des ouvrages d'art et des chaussées. Le présent rapport, Partie 2, présente les résultats de la recherche concernant l'évaluation structurale des ponts routiers sous l'effet du trafic, en particulier vis-à-vis de la fatigue. Le second rapport, Partie 1, concerne le trafic et l'effet de celui-ci sur les chaussées. Afin d'en faciliter la lecture, une introduction et des conclusions générales communes aux deux rapports sont incluses dans chacun de ceux-ci.

La suite de ce résumé concerne la partie 2. Pour les structures en acier, la fatigue se trouve être la troisième cause principale d'endommagement ; elle se classe même comme la première cause d'endommagement des ponts en acier. Lors de l'analyse à la fatigue des ponts, quelque soit le matériau, l'approche traditionnelle est celle de la contrainte nominale, laquelle utilise les courbes S-N caractéristiques sous amplitude constante en combinaison avec la règle de cumul linéaire des dommages selon la loi de Miner modifiée. Les courbes S-N caractéristiques, qui décrivent la relation entre la différence de contrainte nominale appliquée, S , et le quantile p de la durée de vie en nombre de cycles, N , sont obtenues à partir d'essais de fatigue à amplitude constante.

Dans les normes, les courbes S-N sont obtenues en ajustant une régression linéaire sur les points expérimentaux de rupture mais cette approche comporte des limitations qui peuvent affecter sensiblement la précision des résultats d'évaluation de ponts existants, mais aussi le dimensionnement à la fatigue des nouveaux ponts. Différents auteurs ont proposé de surmonter les limitations liées aux courbes S-N à amplitude constante en utilisant la méthode "Maximum Likelihood" (ML) pour ajuster un modèle non linéaire avec CAFL aléatoire aux données expérimentales (ruptures et non-ruptures). Néanmoins, cette approche ne donne pas de méthode explicite pour calculer les quantiles p de la courbe S-N à partir des estimations ML des paramètres du modèle. En outre, puisque le modèle ajusté n'est pas linéaire, la comparaison avec les normes existantes ne s'effectue pas directement. Concernant les limitations liées à l'utilisation de la règle de Miner modifiée, de nombreux travaux de recherche ont été effectués pour étudier l'effet des cycles situés au-dessous de la CAFL sur le processus d'endommagement. Cependant, ces travaux ne fournissent que des tendances générales et ne présentent pas une approche statistique rigoureuse permettant de surmonter les limitations de la loi de Miner modifiée.

Cette recherche propose une approche probabiliste rigoureuse pour l'estimation : 1) Des courbes S-N caractéristiques pour des charges de fatigue à amplitude constante et variable ; 2) De la valeur critique du cumul de dommage (règle de Miner). La nouvelle approche probabiliste comprend également une nouvelle méthodologie pour l'estimation de la fiabilité en fatigue des ponts existants. Les contributions originales de cette approche sont : 1) Utiliser des Simulations de Monte-Carlo (MCS) pour déterminer la vraie courbe S-N caractéristique à amplitude constante après avoir estimé les paramètres du modèle avec la méthode ML; 2) Utiliser un nouveau schéma qui combine les méthodes MCS et ML et qui utilise les données expérimentales à amplitude variable pour estimer la pente réduite de la courbe S-N, m_2 , et la valeur critique de cumul de dommage, D_c ; 3) Apporter un éclairage sur les indices de fiabilité des ponts, en définissant une nouvelle méthodologie qui permet de calculer l'évolution de la probabilité de rupture par fatigue (i.e. la durée de vie restante) comme probabilité conjointe de l'événement de dépassement de la CAFL et de l'événement d'accumulation de dommage critique. Finalement, une réflexion quant aux formats de vérification à la fatigue des normes Eurocodes et valeurs des facteurs de sécurité partiels associés a été menée, y.c.

une nouvelle méthodologie pour la re-calibration des facteurs de sécurité partiels des vérifications en fatigue. Même si utilisés dans le cadre de cette étude qu'avec des assemblages soudés, les outils développés ont une validité générale.

L'application à trois cas d'études de l'approche ML-MCS pour l'estimation des courbes S-N à amplitude constante et la comparaison avec les normes actuelles a permis d'identifier une définition inexacte de la résistance en fatigue, en particulier dans la zone à grand nombre de cycles. En outre, l'application à deux cas d'études de l'approche ML-MCS pour l'estimation des paramètres m_2 et D_c et la comparaison avec les normes actuelles a révélé une définition insuffisante du mécanisme d'accumulation de dommage dans l'application de la règle de Miner. L'application de la méthodologie de fiabilité au cas d'étude du pont de la Venoge a montré le conservatisme, mais aussi l'inexactitude des normes Eurocodes dans l'évaluation de l'indice de fiabilité en fatigue, pour une durée de vie nominale de 100 ans. La réflexion quant aux formats de vérification des normes Eurocodes pour deux détails sensibles à la fatigue a montré que l'hypothèse de considérer les mêmes facteurs partiels de résistance pour tous les formats de vérification est inexacte. Le format pour un dimensionnement en fatigue, aussi valable pour l'évaluation d'un pont existant, et basé sur la CAFL n'est pas du côté de la sécurité. De plus amples réflexions différenciant les ouvrages existants des neufs par rapport aux courbes de fatigue et à la re-calibration des facteurs partiels devraient être menées.

Mots clefs : fiabilité en fatigue, courbes S-N, ponts existants, ponts en acier, Simulations de Monte-Carlo, Maximum Likelihood, indice de fiabilité, facteurs partiels de sécurité.

Zusammenfassung

Das Ziel dieses Forschungsprojekts ist der Bereich der Verkehrssimulation mit dem Bereich der strukturellen Leistungsbestimmung zu kombinieren, um anhand angemessenen Verkehrssimulationen die Auswirkung des Schwerverkehrs auf Fahrbahnen und neue oder bestehende Straßenbrücken besser angehen zu können. Die Resultate dieses Forschungsprojekts sind in zwei Teilen aufgeteilt und begegnen sich in einer umfassenden Methodik, die die strukturelle Bewertung der Kunstbauten und Fahrbahnen ermöglicht. Der vorliegende Bericht, Teil 2, stellt die Resultate der strukturellen Bewertung von Straßenbrücken unter Verkehrsbelastung in Bezug auf Ermüdung dar. Der zweite Bericht, Teil 1, bezieht sich auf den Verkehr und dessen Auswirkung auf die Fahrbahnen. Für eine gute Lesbarkeit sind allgemeine gemeinsame Einführung und Schlussfolgerungen in jedem Bericht enthalten.

Diese Zusammenfassung betrifft jetzt Teil 2. Ermüdung ist im Stahlbau der dritte Grund für Schädigung und in Stahlbrücken der erste. In der Ermüdungsanalyse von Brücken wendet man traditionell den Ansatz der Nennspannung an, unabhängig vom Material. In diesem Ansatz werden charakteristische S-N Kurven mit Beanspruchungszyklen konstanter Amplitude in Verbindung mit der linearen Schadensakkumulation der veränderten Miner-Regel benutzt. Charakteristische S-N Kurven, welche die Verbindung zwischen dem aufgetragten Nennspannungsbereich S und dem p -Quantil der Ermüdungslebensdauer in Anzahl von Zyklen N beschreiben, kommen von Ermüdungsversuchen mit Beanspruchungszyklen konstanter Spannungsamplitude.

In den aktuellen Normen werden die S-N Kurven durch lineare Regressionsanalyse von experimentellen Bruchstellen geschätzt. Dies bringt jedoch Einschränkungen mit sich, die die Genauigkeit der Ermüdungsbewertung von neuen und bestehenden Brücken wesentlich betreffen. Mehrere Autoren haben vorgeschlagen, diese Einschränkungen der S-N Kurven mit Beanspruchungszyklen konstanter Amplitude mit der « Maximum Likelihood » (ML) Methode zu überwinden, um ein zufälliges Dauerfestigkeit (CAFL, constant amplitude fatigue limit) nicht-lineares Modell an experimentelle Datenpunkte anzupassen (Versagen und Nicht-Versagen). Dennoch gibt dieser Ansatz kein explizites Verfahren, um die p -Quantile der S-N Kurven anhand geschätzten ML Modellparametern zu berechnen. Dazu kommt, dass das angepasste Modell nicht-linear ist, sodass kein direkter Vergleich mit den bestehenden Normen gemacht werden kann. Bezüglich den Einschränkungen in Bezug auf die veränderte Miner-Regel sind zahlreiche Forschungsarbeiten durchgeführt worden, um die Schadenswirkung der Zyklen unter der CAFL zu untersuchen. Allerdings bringen diese Arbeiten nur generelle Tendenzen auf und bieten keinen strengen statistischen Ansatz für die Überwindung der Einschränkungen der veränderten Miner-Regel.

Dieses Forschungsprojekt bietet einen rigorosen probabilistischen Ansatz zur Einschätzung von 1) den charakteristischen S-N Kurven für die Ermüdungslasten mit Beanspruchungszyklen konstanter und variabler Amplitude ; 2) dem kritischen Wert von Schadensakkumulation (Miner-Regel). Der neue probabilistische Ansatz enthält ebenfalls eine neue Methodik für die Zuverlässigkeitsbewertung für Ermüdung in bestehenden Brücken. Die eigenartige Beiträge dieses Ansatzes sind : 1) der Gebrauch von Monte-Carlo Simulationen (MCS) für die Bestimmung der richtigen charakteristischen S-N Kurve mit konstanter Amplitude durch die Einschätzung der Modellparametern mit der ML Methode ; 2) der Gebrauch eines neuen Schemas, das die MCS und ML Methoden kombiniert und die experimentellen Daten von variabler Amplitude gebraucht, um die reduzierte Steigung der S-N Kurve, m_2 und den kritischen Wert von Schadensakkumulation D_c einzuschätzen ; 3) die Erläuterung zur Zuverlässigkeitsindex von Brücken, indem eine neue Methodik zur Versagenswahrscheinlichkeitsberechnung durch Ermüdung (z.B. verbleibende Lebensdauer) als gemeinsame Wahrscheinlichkeit des überschreitendes Ereignis der CAFL und der kritischen Schadensakkumulation bestimmt wird. Schlussendlich wurde

eine Überlegung zur Methodik von Ermüdungsnachweis in den Eurocodes und zu den Werten der verknüpften Teilsicherheitsfaktoren geführt, die eine neue Methodik für die Neukalibrierung der Teilsicherheitsfaktoren für Ermüdung einschließt. Obwohl die hier entwickelte Methodik nur im Kontext von Schweißverbindungen geführt wurde, ist sie allgemein anwendbar.

Die Anwendung des ML-MCS Ansatzes für die Einschätzung der S-N Kurven mit konstanter Amplitude an drei Fallstudien und der Vergleich mit den aktuellen Normen hat das Feststellen der ingenauen Definition von Ermüdungsfestigkeit, vor allem bei hochzyklischer Ermüdung, ermöglicht. Durch die Anwendung des ML-MCS Ansatzes an zwei Fallstudien für die Einschätzung von Parametern m_2 und D_c und durch den Vergleich mit den aktuellen Normen hat sich herausgestellt, dass die Definition des Schadensakkumulationmechanismus in der Anwendung der Miner-Regel unangemessen ist. Die Anwendung der Zuverlässigkeitsanalyse an die Fallstudie der Venoge Brücke hat Konservatismus sowie die Ungenauigkeit der Eurocodes in der Bewertung des Zuverlässigkeitsindexes im Bereich Ermüdung für eine Nennlebensdauer von 100 Jahren gezeigt. Die Überprüfung des Eurocode Formats für zwei ermüdungsanfällige Details hat erkennen lassen, dass die Annahme, die gleichen Teilwiderstandsbeiwerte für alle Überprüfungsformate zu benutzen, ungenau ist. Das Format für Ermüdungsbemessung, was auch für die Bewertung einer bestehenden Brücke gilt und auf Grund der CAFL basiert ist, ist nicht auf der sicheren Seite. Weitere Überlegungen, die neuen von bestehenden Bauwerken in Bezug auf Ermüdungskurven und Re-Kalibrierung von Teilfaktoren unterscheiden, sollten neu durchgeführt werden.

Stichwörter : Ermüdungszuverlässigkeit, S-N Kurven, bestehende Brücken, Stahlbrücken, Monte-Carlo Simulationen, Maximum Likelihood, Zuverlässigkeitsindex, Teilsicherheitsfaktoren.

Sommario

La fatica é la terza causa di danno delle strutture in acciaio mentre si colloca al primo posto nella classifica delle cause di danno dei ponti in acciaio. L'analisi a fatica dei ponti in acciaio richiede la verifica delle giunzioni saldate. L'approccio tradizionale di verifica a fatica delle giunzioni saldate é quello della tensione nominale, che consiste nell'utilizzare le curve caratteristiche S-N per carichi ad ampiezza costante insieme alla regola di cumulo lineare del danno, secondo la regola di Miner modificata. Le curve S-N, che descrivono la relazione tra la differenza nominale di tensione applicata, S , e il quantile p della vita a fatica in numero di cicli, N , sono ottenute mediante prove sperimentali a fatica effettuate con carichi ad ampiezza costante.

Nelle norme attuali le curve caratteristiche S-N sono ottenute mediante regressione lineare sull'insieme dei punti sperimentali di rottura. Quest'approccio é caratterizzato da diversi limiti: 1) I punti di non-rottura non sono presi in considerazione; 2) Il limite di fatica ad ampiezza costante (CAFL) viene fissato arbitrariamente ad un determinato numero di cicli; 3) Le curve S-N caratteristiche vengono calcolate sulla base della dispersione della vita a fatica a 2 milioni di cicli, con conseguente difetto di precisione nella zona ad alto numero di cicli. Inoltre la regola di Miner modificata é caratterizzata dai seguenti limiti: 1) L'utilizzo della curva S-N con una pendenza ridotta, m_2 , per cicli con differenza di tensione inferiore alla CAFL, non é stato sufficientemente giustificato mediante l'uso di risultati sperimentali; 2) L'utilizzo di un valore critico del danno accumulato uguale a 1.0 potrebbe portare ad una valutazione inesatta degli effetti legati alla storia dei carichi. I limiti menzionati sopra influenzano fortemente l'accuratezza dei risultati allorché le curve S-N caratteristiche sono utilizzate insieme alla regola di Miner per la verifica a fatica di ponti esistenti o per la progettazione a fatica di nuovi ponti.

Diversi autori hanno proposto di risolvere i problemi legati all'utilizzo delle curve S-N proposte nelle norme, utilizzando il metodo di Massima Verosimiglianza per effettuare il fitting di un modello lineare con limite di fatica aleatorio su un insieme completo di punti sperimentali (che include sia punti di rottura che di non-rottura). Tuttavia quest'approccio non fornisce un metodo esplicito per calcolare i quantili p della vita a fatica a partire dalle stime di massima verosimiglianza dei parametri del modello S-N. Inoltre, poiché il modello S-N di regressione non é lineare, non é possibile effettuare un confronto diretto con le curve S-N delle norme attuali. Riguardo le limitazioni legate all'utilizzo della regola di Miner modificata, esistono numerosi studi di ricerca focalizzati sull'effetto dei cicli con differenza di tensione inferiore al limite di fatica sul cumulo di danno. Tuttavia questi studi danno solo delle indicazioni qualitative e non forniscono un approccio statistico rigoroso che permetta di superare le limitazioni legate all'utilizzo della regola di Miner modificata.

Questa tesi propone un nuovo metodo probabilistico che permette di stimare: 1) Le curve caratteristiche S-N per carichi di fatica ad ampiezza costante e variabile; 2) Il valore critico del danno accumulato da utilizzare nella regola di Miner. Il nuovo metodo comprende anche: 1) Una nuova metodologia per la verifica a fatica dei ponti esistenti; 2) Una nuova metodologia per la ricalibrazione dei fattori parziali di sicurezza a fatica. Il contributo originale di questa tesi consiste nei seguenti punti: 1) Utilizzo del metodo Monte Carlo per la stima delle curve caratteristiche S-N dopo aver determinato i parametri del modello SN mediante il metodo di Massima Verosimiglianza; 2) Utilizzo di un nuovo schema che combina il metodo Monte Carlo e il metodo di Massima Verosimiglianza e che utilizza i risultati sperimentali delle prove a fatica effettuate con carichi ad ampiezza variabile, per stimare la pendenza ridotta della curva S-N e il valore critico del danno accumulato; 3) Definizione di una nuova metodologia che permette di calcolare la probabilit  di rottura a fatica come probabilit  congiunta di eccedenza del limite di fatica e del cumulo critico di danno, migliorando l'accuratezza della stima degli indici di affidabilit  dei ponti; 4) Miglioramento dell'affidabilit  nella progettazione a fatica dei nuovi ponti mediante la revisione delle norme europee e dei fattori parziali di sicurezza associati alla fatica.

L'applicazione del nuovo metodo per la stima delle curve caratteristiche S-N a tre casi di studio e il confronto con le norme attuali ha svelato una definizione inaccurata della resistenza a fatica, in particolare nella zona ad alto numero di cicli. Inoltre l'applicazione del nuovo metodo per la stima dei parametri della regola di Miner a due casi di studio ed il confronto con le norme attuali ha rivelato una definizione insufficiente del meccanismo di cumulo del danno nell'applicazione della regola di Miner. L'applicazione della metodologia per la valutazione dei ponti esistenti al caso del ponte sulla Venoge ha mostrato l'inesattezza delle norme europee nel calcolo dell'indice di affidabilità a fatica, nel corso della durata di vita nominale di 100 anni. Inoltre la questione legata alla scelta del valore di target dell'indice di affidabilità è stata analizzata per lo stesso caso di studio. La revisione dei format delle norme europee per la progettazione a fatica, per due dettagli tipici, ha mostrato che l'ipotesi di considerare gli stessi fattori parziali di sicurezza per tutti i format di verifica è inesatta e che nelle norme europee attuali il format di progettazione a fatica basato sul limite di fatica risulta fortemente rischioso.

Parole chiave: affidabilità alla fatica, curve S-N, ponti in acciaio, metodo Monte Carlo, metodo di Massima Verosimiglianza, indice di affidabilità, fattori parziali di sicurezza.

1 General introduction

1.1 Problematic

Roads and their bridges are public infrastructures with great asset as well as financial values. In case of malfunction of an infrastructure network, civil society suffers inconveniences in economic terms (attractiveness, competitiveness, jobs, real-estate losses) but also in societal terms (freedom of travel, accessibility, well-being, life quality).

Political decisions, urban developments, at regional or national levels involve transportation networks and infrastructure improvements or modifications as well as more globally planning changes of the transportation system as a whole.

These different reasons imply a continuous reassessment of traffic, of road infrastructures and of bridges equally from an operational perspective and from a safety one. In order to do so, it is necessary to have better assessment tools in the domains of traffic simulations as well as structural assessment of road infrastructure components.

Today's traffic simulation tools allow for a realistic view of a road network behaviour, at different scales of time (future previsions, past traffic or actual) and space (road segment, district, city, region). In these traffic models, heavy vehicles or trucks, apart from the space they occupy on a road, are usually not further considered.

In the domain of bridge structural assessment, being it new or existing ones, several previous research projects [55],[49],[56] have demonstrated the necessity to consider in a realistic manner heavy vehicles traffic. From the point of view of design and degradation of pavements, heavy vehicle traffic also has a predominant role. Traffic analyses and modelling carried out in this study aim specifically at considering properly heavy vehicles in traffic and their effects.

When looking into to bridge safety checks and life estimation, it is the fatigue computations which are found to be less accurate, owing to the many assumptions behind them. It has numerous times been demonstrated that the result is influenced by those assumptions made on one hand regarding heavy vehicles traffic on the bridge (note that the equivalent fatigue load model is very conservative) and on the other hand about the damage function (fatigue S-N curve CAFL, damage sum). Analyses and models to be developed in the framework of this study aim at proposing a more effective methodology for assessing fatigue life of individual road bridges (regarding their geometry and loading).

1.2 Objectives of the research project

The main objective of this research project is to combine the two domains of traffic simulation and structural performance determination. This is done by **creating interfaces between the different existing tools** at disposal of engineers regarding road traffic analyses (statistical tools, software for traffic simulations) and structural evaluation indexes modules for bridges (ultimate limit states, including fatigue) as well as for pavements (damage, rutting), similarly to the degrees of compliance from the SIA 269. The behaviour and specific characteristics of the heavy traffic are integrated into databases as well as vehicle behavioural models in order to **establish a procedure for realistic traffic simulations**. For several bridge examples, load histories on them were determined and validated.

Moreover, this research project aims at **improving the methodology for existing bridge structural safety checks, remaining life as well as pavement degradation subjected to traffic loads**. It proposes a new probabilistic approach, allowing for the determination of fatigue curves and the critical damage sum value by reanalysing the experimental results under constant (CA) as well as variable amplitude (VA). For stress

range spectra having a low to very low percentage of cycles above the CAFL, improved ways are proposed for the modelling of fatigue damage evolution. The developed tools have general applicability, but will be applied herein to welded joints.

The different interfaces and methodologies developed in this research project offer **new tools for degradation evolution computations** and thus contribute to decision making in the domain of management of road transportation infrastructures.

1.3 Global approach

The global approach is summarized in Fig. 1. The road data needed to know traffic on examined road sections are taken from two different types of counting devices. On one hand, the data come from the CSACR (Comptage suisse automatique de la circulation routière) which indexes the crossings of all vehicles and, on the other hand, the data are obtained from the WIM (Weigh in Motion) counting network, which gets information regarding heavy vehicles crossings. The raw data files contain information such as the counting station, rebooting of the counting device or recording errors, which need to be “cleaned” before data analysis. Moreover, the raw data types differ from one counting station to another. For these reasons, cleaning and formatting scripts of the data records were created in the operating system MS-DOS (*.bat).

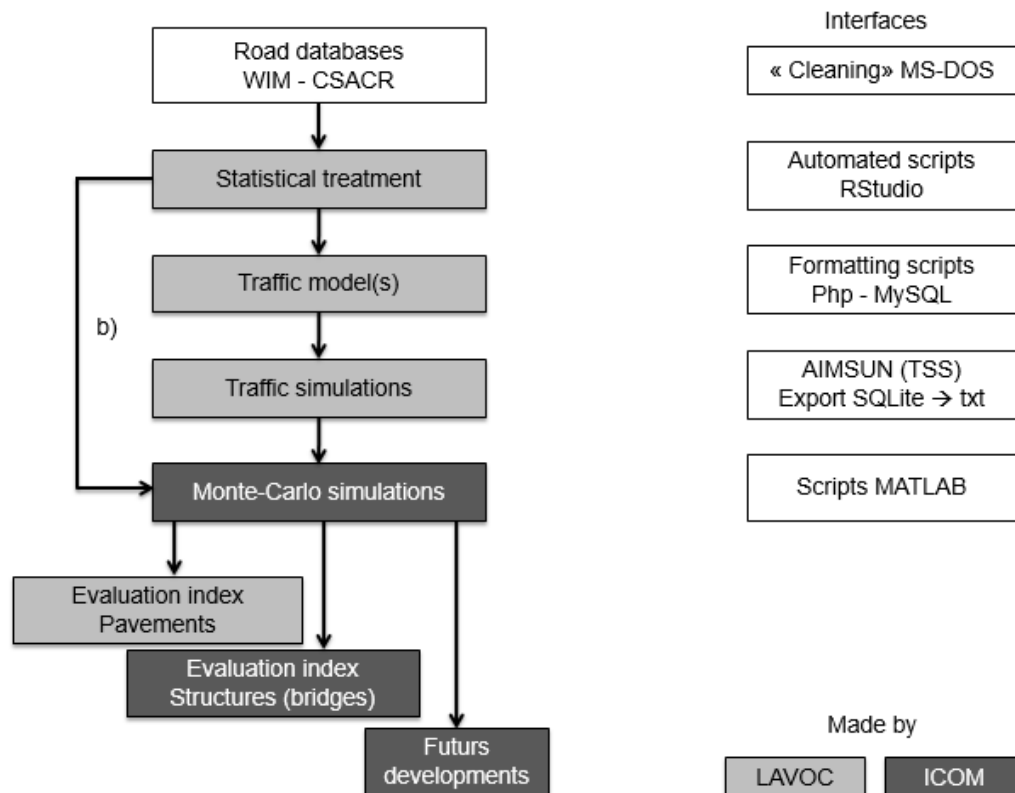


Fig. 1 Study global approach.

The statistical treatment (hourly distribution of traffic, vehicle characteristics) are made with the help of automated scripts developed in the statistical processing software RStudio (computer language R).

Traffic matrices Origin-Destination or local traffic states and the related turning information found from the statistical processing and driver behaviour characteristics as well as of the vehicles are integrated in the traffic simulation software AIMSUN (Advanced Interactive Microscopic Simulator for Urban and Non-Urban Networks) developed by the company TSS (Transport Simulation Systems). This step is made thru

the use of formatting scripts developed in programming language PHP (imperative object-oriented language) linked to MySQL databases. Various models and traffic simulations are made with the help of AIMSUN.

The information generated during traffic simulations are exported via the SQLite tool (integrated into AIMSUN) for given points in the modelled network, a tool which automatically creates databases compatible with the SQL language (management of relational databases). The different information gathered finalize the road traffic treatment part and are then transformed into text format (*.txt) in order to proceed to the structural evaluation part. Note that, for simple traffic cases, it is also possible to shortcut the traffic simulation part (see Fig. 1, plan (b)) and make a direct use of the statistically treated road data (hourly distribution of traffic, vehicle characteristics) to generate a series of vehicles moving at a constant speed on a bridge or pavement section.

Some heavy vehicles statistical characteristics cannot be integrated in a coherent manner in the traffic simulation software AIMSUN (i.e. loads per axles, total weight) due to their specific statistical laws types (non-Gaussian laws). In order to enrich each vehicle crossing with these characteristics, and determine the action effects, Monte-Carlo simulations are made using MATLAB scripts.

From those Monte Carlo simulations, one can estimate on one hand the pavement damage (rutting, residual life, eventual reinforcement) (Report, Part 1) and, on the other side, the structural safety as well as the safety against fatigue of bridges (Report, Part 2).

1.4 Study case: methodology

The study of a specific case is done with a methodology that follows the logic of the global approach explained above, with the clarifications contained herein. In the global phase of definition of the project, one first studies the localization and situation of the selected road/bridge sections. From the structural point of view, the generic typology of the bridge is defined (type of materials used, type of pavement, dimensions, composition). From the road analysis point of view, the type of traffic crossing the bridge is determined, as well as the road geometry in alignment, cross section and layout.

Then the more detailed phases start. Firstly, the study perimeter is defined so that the different elements influencing the traffic be accounted for in the road analysis (influence from road signs, traffic lights, priority rules, roundabouts, generators of traffic such as industries, shops, industrial zones, housing districts, neighbouring villages, ...). Secondly, one also determines the catalogue of the various vehicle classes that will be considered as well as the necessary assumptions for the traffic and the behaviour models. To this end, one defines a time frame in relation to the statistical treatment of the traffic data. Thirdly, the bridge/road under study is modelled in order to get the sections, the details to be checked and for bridges the corresponding transfer functions (i.e. influence lines, functions relating loads to stresses, or limit state function for reliability analysis). For the road pavement, several examples are given in Part 1 report (traffic). Regarding the modelling of a structure, it is presented for one study case (Venoge bridge, A1) but is considered as being part of the expertise domain of the engineer in charge of the project, and thus does not constitute the topic of this research.

Finally, the traffic and Monte-Carlo simulations can start. The emphasis is put on fatigue limit state, the S-N curves, which express the fatigue strength of the detail to check, and the limit state function for reliability analysis, being redefined. They use a new probabilistic approach which is presented in the Part 2 report (fatigue).

2 Introduction

2.1 Background: fatigue of joints

Fatigue is defined as a mechanism of failure of a structural member due to the formation and growth of cracks under the effect of repeated stresses [34].

First studies on fatigue date back to the second half of 19th century, when Wöhler published the results of his fatigue tests on railway axles [87]. Studies on welded joints started only after the second World War, when welding started to become a major production process.

Fatigue cracks appear near the regions where the local stress is much higher than the nominal stress; these regions are called stress concentrators. When the stress concentrators lead to fatigue cracking they are called crack initiators.

Fatigue can be classified according to: 1) Type of crack initiators: holes, notches, material structure discontinuities, welds; 2) Form in which fatigue occurs: mechanical, thermomechanical, corrosion, creep, fretting, rolling contact [79]; and 3) Duration: low-cycle, high-cycle. Fatigue verification of road steel bridges under the long term effect of traffic asks for consideration of high-cycle mechanical fatigue in particular welded joints as they are often the weaker details in fatigue compared to the bolted and riveted ones.

The prevention of fatigue failure requires designing fatigue sensitive structure with a sufficient fatigue strength. However, fatigue strength is not a constant material property, like limit of elasticity for example. For an assigned detail geometry the fatigue strength can be calculated by using linear elastic fracture mechanics (LEFM) [86]. Use of LEFM includes several assumptions on the cracking process, which could largely affect the final results.

For this reason, the fatigue verification of structures subjected to variable amplitude (VA) loadings is generally performed by using constant amplitude (CA) S-N curves (which relate the nominal applied stress range, S , to the number of cycles to failure, N), in combination with a cycle-counting algorithm and a damage accumulation rule. The fatigue verification problems can be simplified, by keeping all the stress range cycles below the constant amplitude fatigue limit (CAFL): in this case only the combination of CA S-N curves with a cyclecounting algorithm is needed. However, this type of fatigue verification is over-conservative and may lead to un-necessary heavy structures (in case of fatigue design of new structures), or to un-necessary and expensive inspections (in case of fatigue assessment of existing structures).

2.1.1 CA S-N curves

CA S-N curves are estimated by means of CA experimental fatigue tests on given details, which include plain members, bolted joints, rebars, welded joints,.... Due to the randomness of fatigue life, statistical evaluation of experimental results is needed. In order to estimate the median S-N curve the experimental failure points are generally fitted with following linear regression:

$$\log(N) = C + m \cdot \log(S) \quad , \text{ for } S > \text{CAFL} \quad (\text{Eq. 1})$$

where C and m are material constants¹. The complete nomenclature is provided in Appendix I.

¹ Contrary to common practice, in this thesis negative m is used in order to have correct definition of S-N curve in analytic geometry.

Characteristic S-N curve is then estimated by translating the median S-N curve proportionally to the fatigue life scatter at $2 \cdot 10^6$ cycles.

LEFM can be used to show the validity of Equation 1 [85].

The fatigue failure process can be divided in three phases: 1) Crack initiation; 2) Stable crack propagation; and 3) Unstable crack propagation, which is very limited in time. The stress field near the crack tip can be expressed by using the stress intensity factor:

$$K = M_k \cdot S_n \cdot \sqrt{\pi a} \quad (\text{Eq. 2})$$

where S_n is the nominal stress, M_k is the geometrical correction factor, and a is the crack size.

In 1963 Paris and Erdogan [62] proposed to relate the stress intensity range to the crack propagation rate according to the idea that fatigue crack propagation phenomenon must depend on the stress field around the crack tip:

$$\frac{da}{dN} = C_1 (\Delta K)^{-m} \quad (\text{Eq. 3})$$

where C_1 is a material constants and ΔK is the difference between the stress intensity factor at the upper and lower limit stresses of the loading cycle.

In welded joints and in the finite life region, the crack initiation phase is almost non existent (fatigue cracks usually initiate at weld toe, where sharp defects can be regarded as small cracks) and the entire fatigue life can be taken by the stable propagation phase. Under this assumption, the Paris Law (Equation 3) can be applied to all fatigue life of welded joints; thus inserting Equation 2 in Equation 3, gives:

$$\int_{a_1}^{a_2} \frac{da}{\left(M_k \sqrt{(\pi a)}\right)^m} = C_1 \cdot S^{-m} \cdot N \quad (\text{Eq. 4})$$

where S is the nominal stress range.

Equation 4 confirms that, for a given type of joint and constant values of a_1 and a_2 , the relationship between S and N is linear in the $\log(S-N)$ plane:

$$S^{-m} \cdot N = \text{constant} \quad (\text{Eq. 5})$$

Even if could be based on other models, such as Manson-Coffin [51], the same S-N curve model is also used for non-welded joints, see for example Eurocode EN1993-1-9 [2]. The constant amplitude fatigue limit (CAFL) adds the following condition to the above : $N = \infty$ if $S \leq \Delta\sigma_D$. In LEFM, it can similarly be modeled in a simplified way by

adding a threshold to Equation 3, i.e. $\frac{da}{dN} = 0$ if $\Delta K \leq \Delta K_{th}$. It is important to add that

crack initiation mechanisms can become significant for stress ranges near to the CAFL, in the high and very high cycle fatigue regions.

2.1.2 Cycle counting methods

Cycle counting methods allow breaking down a complex load history in simple load cycles, which can be easily applied to CA S-N curves for fatigue evaluation. A realistic counting algorithm should take in account all the stress strain closed hysteresis loops, which are contained in the loading history.

In 1974 Endo et al. [26] proposed the rain-flow method. Its name originates from the fact that the stress history is expressed by using rain-flow on a pagoda roof while the stress ranges are counted by the ranges of the rain-flow. This method allows to count all the closed hysteresis loops and it is by far the most largely used cycle counting algorithm. Various other methods have been also proposed (peak method [3], range method, range pair method [71]) but they are affected by the limitation of giving inaccurate evaluation of closed hysteresis loops.

2.1.3 Damage accumulation

Damage accumulation rules are used in combination with a fatigue load spectrum and a CA S-N curve, in order to assess fatigue life of a critical detail. The most widely used damage accumulation rule is the Miner's linear rule [58], which predicts the number of cycles to fatigue failure by assuming that the damage caused by one stress range cycle of VA loading is the same as the one caused by the same stress range in CA loading:

$$\sum_{i=1}^{n_{tot}} \frac{n_i}{N_i} = 1 \quad (\text{Eq. 6})$$

where n_i is the number of cycles corresponding to the stress range S_i and N_i is the number of cycles to failure corresponding to the stress range S_i , according to the CA S-N curve.

If the CA S-N curve has been estimated for a fully reversed CA loading, then the mean VA stress has to be corrected; the modified Goodman law [30] is conventionally used for this purpose. The main shortcomings of the Miner's rule are: 1) The fatigue damage given by cycles below the CAFL is not considered; 2) The effects related to VA load history on fatigue damage are not considered as well; these effects are all related to the crack closure mechanism [24].

Many modifications of Miner's rule have been proposed in order to overcome its first shortcoming [45] (see Fig. 2). The most largely used rule is the Haibach's rule [37], which considers a S-N curve with a reduced slope, $m_2 = 2m_1 + 1$ below the CAFL. This rule gives a good estimation of fatigue life when the percentage of stress range cycles below CAFL is large.

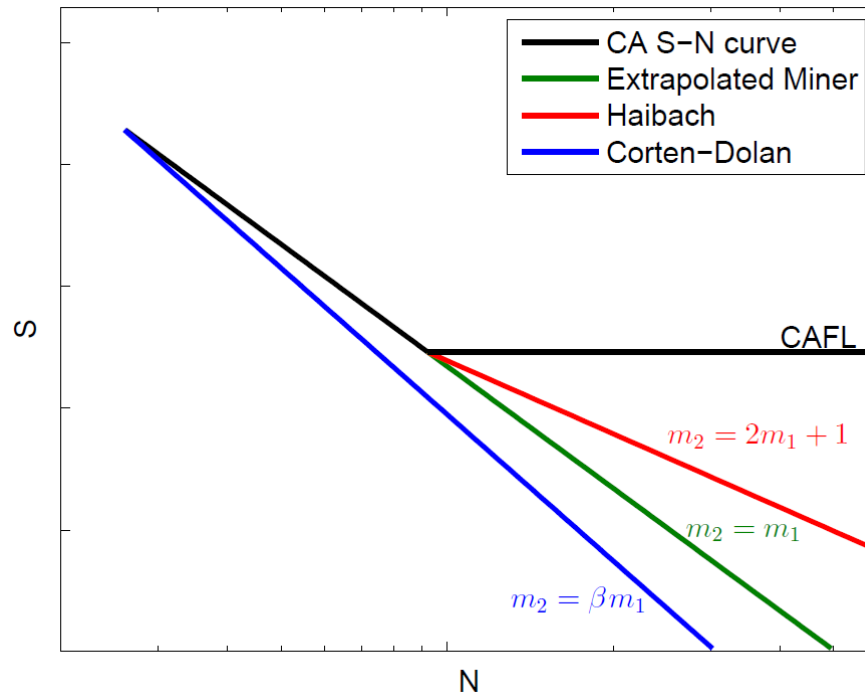


Fig. 2 VA S-N curves.

The second shortcoming of Miner's rule is still an unclear point. In most cases the Miner's rule (Equation 6) is not valid because load sequence effects cause significant deviation from the CA crack propagation rate, resulting in a value of the critical damage sum at failure, D_c , which is not equal to unity. Many authors have analysed the influence of load sequence on Miner's rule in welded joints, but without providing a rigorous method for its modification. This topic is discussed more in details in the Introduction of Chapter 4.

2.2 Problem statement

Reliable fatigue verification of road steel bridges and in particular of welded joints, requires realistic assessment of fatigue resistance S-N curves as well as realistic consideration of damage accumulation.

In current standards [28], [42], [4], characteristic CA S-N curves are determined by fitting a linear regression to experimental failure points, disregarding run-outs and somewhat arbitrarily fixing the CAFL at a given number of cycles. Moreover, the fatigue life scatter in the high cycle fatigue (HCF) region ($N > 5 \cdot 10^6$) is not modeled properly. CA characteristic S-N curves are conventionally used in combination with modified Miner's rule, which results in un-realistic consideration of load history effects.

Limitations mentioned above may heavily affect the accuracy of results when the combination of characteristic CA S-N curves with modified Miner's rule (Miner's rule with Haibach's modification) is used in fatigue design of new bridges or in fatigue assessment of existing bridges.

Linear regression analysis and modified Miner's rule do not provide accurate definition of the stochastic VA S-N model (which includes probability distributions of S-N curve parameters and of critical damage sum parameters). This limitation can lead to inaccurate computation of reliability indexes when fatigue reliability analysis of existing bridges is carried out for: 1) Assessment of the remaining fatigue life; 2) Planning of inspections; and 3) Planning of repair interventions.

In structural codes the fatigue design is carried out by using design equations which compare resistance and loading terms. In design equations the uncertainty on loading and resistance terms is considered in a semi-probabilistic model, further simplified, by introducing partial safety factors. Limitations related to VA S-N model may lead to inaccurate calibration of fatigue partial safety factors, resulting in inaccurate design of new bridges.

Based on the discussion above it is clear that a rigorous statistical approach is needed for estimation of stochastic VA S-N models from experimental CA and VA fatigue test results. Advances in VA S-N model estimation will give more insight into realistic fatigue reliability indexes and into reliable calibration of partial safety factors for fatigue design.

2.3 Objectives

The main aim of this study is to provide a novel probabilistic approach which allows to redefine both S-N curves and critical damage sum parameter, by using experimental CA and VA fatigue test results.

The objectives are as follows:

- To provide a new probabilistic approach for estimation of CA S-N curve random parameters and for assessment of true p -quantiles of CA S-N curves;
- To provide a new probabilistic approach for estimation of VA S-N stochastic model parameters;
- To provide a new framework for fatigue reliability assessment of existing bridges, in which the probability of having fatigue failure is computed as joint probability of having critical damage accumulation and CAFL exceedance;
- To provide a new framework for calibration of partial safety factors for fatigue design.

Note that the last objective has been added as it was part of Dr. D'Angelo doctoral thesis [20] but was not initially included in the objectives of this work. It is included as it is considered relevant to the problematic at large. This report shows some differences with the doctoral thesis as it clarifies some points related to practical application and contains an additional section on structural indexes computation.

2.4 Structure of the report

The structure of this report is shown in Fig. 3, which also illustrates the relationships between topics. The state of art of each topic is presented at beginning of the related Chapter.

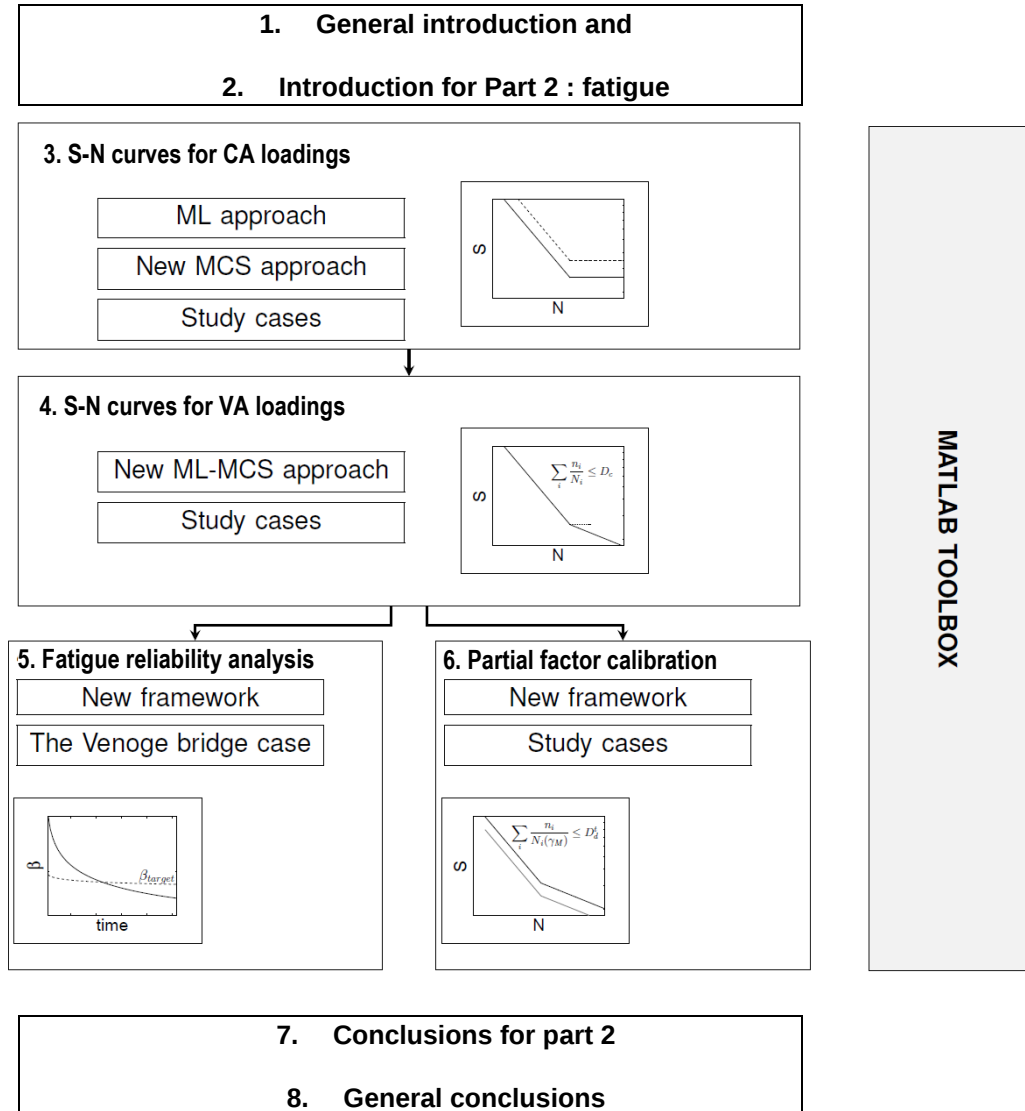


Fig. 3 Structure of the report.

3 S-N curves for constant amplitude loadings

3.1 Introduction

Traditional fatigue analysis of components under CA and VA loadings is based on the nominal stress approach, where the verification can be based on CAFL or on damage accumulation. Within the two verifications above, the choice of CA fatigue strength curve is of primary importance and thus is the topic of this chapter. The developments that are made have general applicability, even if they will be herein applied to welded joints.

Fatigue strength of welded components is expressed in terms of characteristic S-N curves, which give the fatigue life, N , at each stress range, S , for a certain survival probability, P_s (typically 95%), on the basis of a certain confidence level on the S-N curve parameters (typically 75%). S-N curves are based on experimental tests; however, due to the inherent randomness in the fatigue life, a statistical treatment of experimental data-sets is required.

Median S-N curve² ($P_s = 50\%$) in current Eurocode standards [1] is derived by fitting a linear regression which has a slope of $m_1 = -3$ to the failure data, using a least squares (LS) analysis procedure. Characteristic S-N curve is then derived by translating the median S-N curve on the lower 5% prediction bound, at 2 million cycles; the CAFL is arbitrarily assumed at 5 million cycles³. The linear regression used disregards run-out data and arbitrarily fixes the CAFL at 5 million cycles, which is unrealistic and neglects important information from experimental observations. Further, to deduce the characteristic S-N curve, one shifts the median curve and thus assumes the fatigue data-set to be homoscedastic. The limitation of LS analysis procedure, connected to run-outs omission in estimation of S-N curves, may be overcome by using Maximum Likelihood (ML) method.

In [77], Spindel and Haibach used ML method to estimate S-N curves for a generic fatigue data-set containing both failures and run-outs. CAFL was not included in the model and it was assumed at 2 million cycles.

In [60], Nelson used ML method to include run-outs in S-N curve estimation of a nickel base super-alloy; Nelson modeled the mean and the standard deviation of the fatigue life as functions of the stress range but the CAFL was not included in the model.

In [61] Nelson proposed several models which included the CAFL.

In [63] Pascual et al. proposed a 5-parameter random fatigue limit (RFL) model that fitted a non-linear S-N curve with a random CAFL to a complete fatigue data-set, using ML method. Median S-N curve and standard deviation parameters expressed the randomness in the fatigue life, while the randomness in the CAFL was expressed by the location and the scale parameters of a generic location-scale probability family. Median and characteristic S-N curves with profile likelihood ratio (PLR)-based confidence bounds were generated using a ML approach.

² Median is preferred here instead of Mean, as different probability distributions than Normal will be used to represent fatigue life randomness.

³ This value has been originally taken as a compromise between $2 \cdot 10^6$, proven to be too low in some studies, and 10^7 , seen as too conservative even if shown in some studies. ECSS and later EN 1993-1-9 S-N curves were established for the design new structures, not for the assessment of existing ones. Indeed, simplicity (i.e. having a set of parallel curves) was chosen over more accuracy (different curves slopes, nb of cycles corresponding to CAFL, ...) [40].

In [47], Lassen et al. fitted RFL model from Pascual to two experimental data-sets of a plate with fillet welded attachments. Lassen et al. observed that: 1) The RFL method-based 90% confidence interval of the median of CAFL distribution did not include the Eurocode CAFL (median value); 2) The RFL method-based 90% confidence interval of the median of CAFL included the BS5400 CAFL (median value) and the RFL model-based 0.025 quantile S-N curve was in good agreement with BS5400 F-class S-N curve for stress ranges higher than 150MPa; 3) At stress ranges lower than 150 MPa the non linear RFL model-based 0.025 quantile S-N curve predicted fatigue lives that were up to 10 times longer than the predictions made by the F-class S-N curve. Confidence level of 0.025 quantile S-N curve was not indicated.

In Part 3 of the JCSS PMC [43], a ML-based approach is recommended for the estimation of SN-curves parameters: a linear S-N model is fitted to both failure and run-out experimental points and uncertainty related to the parameter fit is estimated by using the Hessian of the log-Likelihood function. It is not specified if the CAFL has to be considered as a random variable whose parameters are included in the model parameter vector or if it has to be considered as a deterministic value which is arbitrarily assumed at a fixed number of cycles. Recommended approach also does not specify how to relate the uncertainty on the parameter fit to the uncertainty on the fatigue life quantiles.

The existence of the CAFL is still an object of dispute; Sonsino [75] proposes a CA bi-linear S-N curve where stress range cycles below the knee point are accounted with a slope $m'_1 = -22$. The knee point is arbitrarily fixed at $N = 10^7$. The slope $m'_1 = -22$ is also arbitrarily fixed in order to have 10% constant stress range decrease with respect to the $\log(N)$ axis for each order of magnitude. The lack of the experimental results in the HCF region and the related high scatter of fatigue life in this region does not allow to provide experimental validation to the CA bi-linear S-N model. For this reason the choice of a CA S-N model including random CAFL seems to be the most realistic choice allowing for correct interpretation of experimental results in HCF region.

The RFL model proposed by Pascual, allows to overcome limitations of current Eurocode approach but it does not give explicit method to derive the true p – quantile S-N curve from ML estimate of CA S-N stochastic model parameters. Furthermore, the RFL model-based median S-N curve is not linear and direct comparison with current standards is not straightforward.

In this Chapter a novel approach, based on ML method and on Monte-Carlo Simulations (MCS) method, is proposed for estimation of median and characteristic S-N curves under CA fatigue loadings. A linear S-N model is fitted to a complete fatigue experimental data-set, using ML method. CAFL is modeled as a random variable whose parameters are included in the CA S-N model parameter vector. The covariance matrix of the model parameter vector is estimated by inverting the observed Fisher information matrix. The true 0.05 quantile S-N curve (corresponding to a probability of failure, $P_f = 5\%$) is estimated by using MCS method. The novel contribution of this approach is summarized in following points:

- Linear S-N model with random CAFL is fitted directly to experimental data;
- p – quantiles of S-N curve are estimated using MCS method. MCS-based p – quantiles correspond to the true $(100 \cdot p\%)$ probability of failure and not to the nominal $(100 \cdot p\%)$ probability of failure with α – confidence level (as for the case of RFL model);
- Linearization of ML-MCS-based characteristic S-N curves allows to make direct comparison with current standards.

The ML-MCS approach which is presented in this work was implemented in CAestimation and CAquantiles modules of Matlab [53] Toolbox TB1 (see Appendix III).

The Chapter is organized as follows:

- in Section 3.2 the statistical analysis procedure for estimation of characteristic S-N curves, based on the Eurocode 3 background documentation, is recalled;
- in Section 3.3 the ML-MCS approach for estimation of characteristic S-N curves is presented;
- in Section 3.4 the application of the ML-MCS approach on three study cases is presented;
- in Section 3.5 results of analysis of considered study cases are presented;
- in Section 3.6 results are discussed.

3.2 Eurocode statistical approach for estimation of S-N curves

In this Section an overview is given of the statistical estimation of characteristic S-N curves, according to Eurocode 3 commentary document [31].

The median S-N curve is estimated by carrying out a Least Square (LS) fit analysis on the experimental data-set $(x_i, y_i)_{i=1, \dots, n_{fail}}$, where x_i is the base 10 logarithm of the applied nominal stress range, y_i is the base 10 logarithm of the observed number of cycles to failure, and n_{fail} is the total number of failed specimens (see Fig. 4 (a)). Observed results having a fatigue life at failure higher than $5 \cdot 10^6$ cycles are considered as run-outs and hence they are not included in the analysis.

The equation of the median S-N curve is the following:

$$\log(N) = m_{0_{LS}} + m_{1_{LS}} \cdot \log(S) \quad (\text{Eq. 7})$$

where $m_{0_{LS}}$ and $m_{1_{LS}}$ are the LS estimators of m_0 and m_1 .

The LS estimator of the log-strength at $2 \cdot 10^6$ cycles is indicated as $x_{c_{LS}}$ (see Fig. 4 (a)).

By putting $N = 2 \cdot 10^6$ in the Equation 7, it follows:

$$\log(2 \cdot 10^6) = m_{0_{LS}} + m_{1_{LS}} \cdot x_{c_{LS}} \quad (\text{Eq. 8})$$

The 95% hyperbolic lower prediction bound⁴ of $Y(x_{c_{LS}})$ is defined as follows (see Fig. 4 (b)):

$$Y_c = \log(2 \cdot 10^6) + t_{.05, n_{fail}-2} \cdot S \sqrt{1 + \frac{1}{n_{fail}} + \frac{(x_{c_{LS}} - \bar{x})^2}{S_{xx}}} \quad (\text{Eq. 9})$$

where:

- $t_{.05, n_{fail}-2}$ is the inverse of Student's t cdf for probability equal to 5% and $(n_{fail} - 2)$ degrees of freedom;
- S is the sample standard deviation;

⁴ See Appendix II.4

- $\bar{x} = (\sum_{i=1}^{n_{fail}} x_i) / n_{fail}$;
- $S_{xx} = (\sum_{i=1}^{n_{fail}} (x_i - \bar{x})^2)$.

By inserting Equation 8 in Equation 9, it follows:

$$Y_c = m_{0LS} + m_{1LS} \cdot x_{cLS} + t_{.05, n_{fail}-2} \cdot S \sqrt{1 + \frac{1}{n_{fail}} + \frac{(x_{cLS} - \bar{x})^2}{S_{xx}}} \quad (\text{Eq. 10})$$

The characteristic S-N curve is determined by translating the median S-N curve in the point $(x_c; Y_c)$, as shown in Fig. 4 (c).

The equation of the characteristic S-N curve is the following:

$$\log(N) = Y_c - m_{1LS} \cdot x_{cLS} + m_{1LS} \cdot \log(S), \quad N \leq 5 \cdot 10^6 \quad (\text{Eq. 11})$$

The CAFL is assumed to start at $5 \cdot 10^6$ cycles (see Fig. 4 (d)).

By inserting the Equation 10 in Equation 11, the characteristic S-N curve may be written as:

$$\log(N) = m_{0LS} + t_{.05, n_{fail}-2} \cdot S \sqrt{1 + \frac{1}{n_{fail}} + \frac{(x_{cLS} - \bar{x})^2}{S_{xx}}} + m_{1LS} \cdot \log(S), \quad N \leq 5 \cdot 10^6 \quad (\text{Eq. 12})$$

The characteristic value of fatigue resistance at $2 \cdot 10^6$ cycles can be determined by putting $N = 2 \cdot 10^6$ in Equation 12:

$$x_c = \frac{\log(2 \cdot 10^6) - m_{0LS} - t_{.05, n_{fail}-2} \cdot S \sqrt{1 + \frac{1}{n_{fail}} + \frac{(x_{cLS} - \bar{x})^2}{S_{xx}}}}{m_{1LS}} \quad (\text{Eq. 13})$$

In the Eurocode 3 commentary document [31], the characteristic value of fatigue resistance at $2 \cdot 10^6$ cycles, x_c , is erroneously indicated as *0.95 lower confidence bound*: in fact it has to be considered as *0.95 lower prediction bound* due to the term 1 in the square root of Equation 13.

For comparison, in IIW recommendations [42] the characteristic S-N curve is obtained by translating the median S-N curve at the 0.05 quantile of the Y normal distribution, by using the lower bound of the 0.75 two-sided confidence interval of $E(Y)$ and the upper bound of the 0.75 two-sided confidence interval of standard deviation. The median S-N curve is obtained by carrying out a LS analysis of experimental failure points. The CAFL is assumed to start at 10^7 cycles. The difference between the confidence interval concept and the prediction interval concept is discussed more widely in Appendix II.

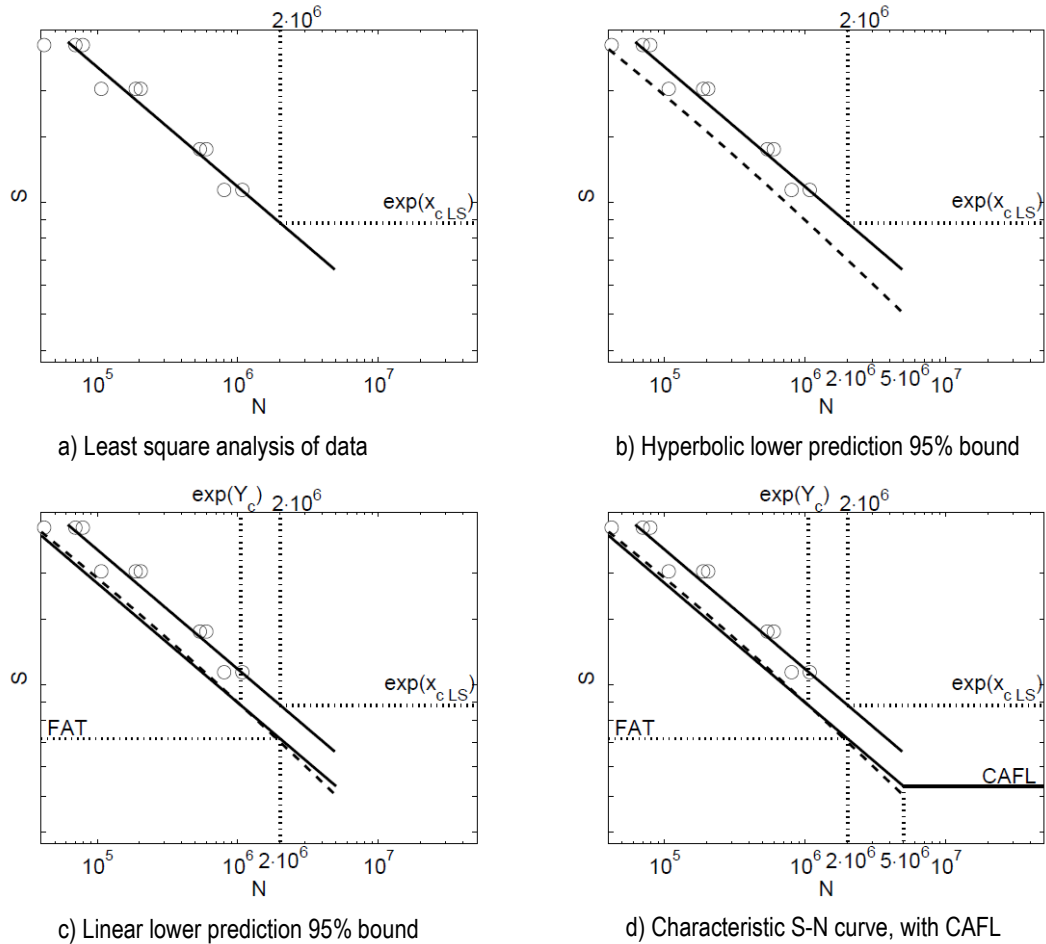


Fig. 4 Eurocode 3 commentary document statistical approach for estimation of S-N curves.

3.3 New probabilistic approach

3.3.1 CA S-N stochastic model

The CA S-N stochastic model describes the relationship between the nominal applied stress range, S , and the number of cycles to failure, N , under CA loadings (see Fig. 5):

$$Y = \frac{m_0 + m_1 X}{H(X - V)} + \epsilon(0, \exp(\sigma)) \quad (\text{Eq. 14})$$

where:

- Y is the natural logarithm⁵ of the number of cycles, N ;
- m_0 is the intercept of the S-N curve in the log(S-N) plane;
- m_1 is the slope of the S-N curve;
- $H(\cdot)$ is the unit step function;
- V is the natural logarithm of the CAFL;

⁵ Natural logarithms are preferred since they are mathematically less heavy to manipulate.

- $\mathcal{E}$ is the error term, which is assumed to be normally distributed with mean equal to 0 and standard deviation equal to $\exp(\sigma)$.

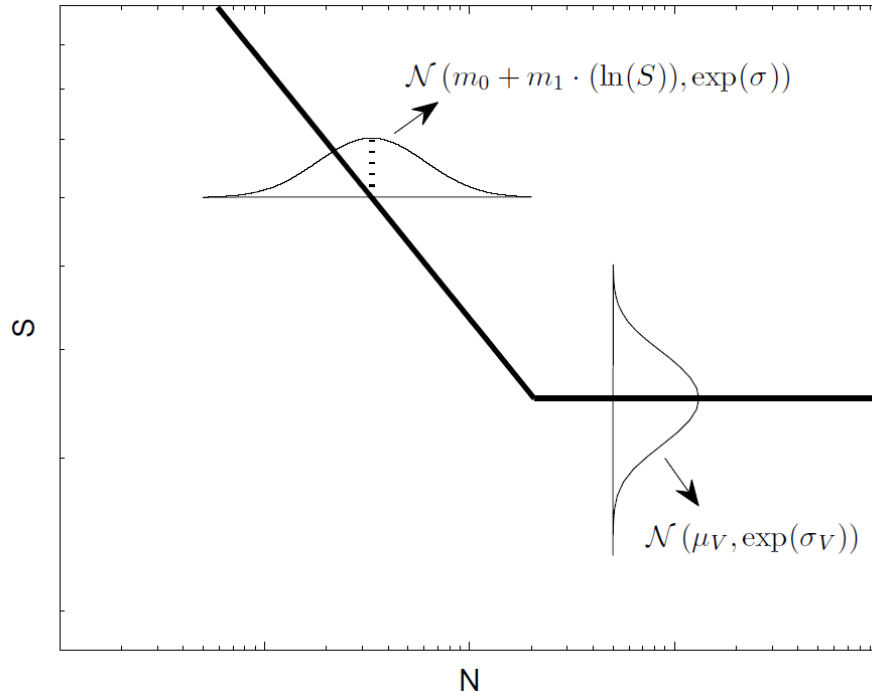


Fig. 5 Median CA S-N curve with model parameters (case of $V = N$).

$Y|V$ is a random variable following a Normal distribution, with location and scale parameters which are respectively:

$$\mu_{Y|V} = \frac{m_0 + m_1 X}{H(X - V)} \quad (\text{Eq. 15})$$

$$\sigma_{Y|V} = \exp(\sigma) \quad (\text{Eq. 16})$$

V is a random variable following a Normal or an Extreme Value (EV) distribution with location parameter equal to μ_V and scale parameter equal to $\exp(\sigma_V)$. If V has a Normal distribution with parameters μ_V and $\exp(\sigma_V)$, then the CAFL has a log-Normal distribution with parameters μ_V and $\exp(\sigma_V)$; instead if V has an EV distribution with parameters μ_V and $\exp(\sigma_V)$, then the CAFL has a Weibull distribution with parameters $a = \exp(\mu_V)$ and $b = \frac{1}{\exp(\sigma_V)}$. The probability density function (pdf) of V is given by:

$$f_V(v, \mu_V, \sigma_V) = \frac{1}{\exp(\sigma_V)} \phi_V \left(\frac{v - \mu_V}{\exp(\sigma_V)} \right) \quad (\text{Eq. 17})$$

where $\phi_V(\cdot)$ is either the standardized Normal pdf or the standardized EV pdf. Conditioned on a fixed value of $V \leq x$, $Y|V$ has a conditional pdf given by:

$$f_{Y|V}(m_0, m_1, \sigma; y, x, v) = \frac{1}{\exp(\sigma)} \phi_{Y|V} \left(\frac{y - \left[\frac{m_0 + m_1 x}{H(x - v)} \right]}{\exp(\sigma)} \right) \quad (\text{Eq. 18})$$

where $\phi_{Y|V}$ is either the standardized normal pdf:

$$\phi(z) = \frac{\exp\left(-\frac{1}{2}z^2\right)}{\sqrt{2\pi}} \quad (\text{Eq. 19})$$

or the standardized EV pdf:

$$\phi(z) = \exp(z) \cdot \exp(-\exp(z)) \quad (\text{Eq. 20})$$

The marginal pdf of Y is given by:

$$f_Y(\underline{\theta}; y, x) = \int_{-\infty}^x \frac{1}{\exp(\sigma + \sigma_V)} \phi_{Y|V} \left(\frac{y - \left[\frac{m_0 + m_1 x}{H(x - v)} \right]}{\exp(\sigma)} \right) \phi_V \left(\frac{v - \mu_V}{\exp(\sigma_V)} \right) dv \quad (\text{Eq. 21})$$

where $\underline{\theta}$ is the model parameter vector. f_Y expresses the probability to have a failure at $Y = y$, for an applied log-stress range equal to $\exp(x)$.

Conditioned on a fixed value of $V \leq x$, $Y|V$ has a conditional cdf given by:

$$F_{Y|V}(m_0, m_1, \sigma; y, x, v) = \Phi_{Y|V} \left(\frac{y - \left[\frac{m_0 + m_1 x}{H(x - v)} \right]}{\exp(\sigma)} \right) \quad (\text{Eq. 22})$$

where $\Phi_{Y|V}$ is either the standardized normal cdf or the standardized EV cdf. The marginal cdf of Y is given by:

$$F_Y(\underline{\theta}; y, x) = \int_{-\infty}^x \frac{1}{\exp(\sigma_V)} \Phi_{Y|V} \left(\frac{y - \left[\frac{m_0 + m_1 x}{H(x - v)} \right]}{\exp(\sigma)} \right) \phi_V \left(\frac{v - \mu_V}{\exp(\sigma_V)} \right) dv \quad (\text{Eq. 23})$$

and expresses the probability to have a failure for $Y \leq y$, for an applied log-stress range equal to $\exp(x)$. $1 - F_Y$ expresses the probability to have a failure for $Y > y$ and then to have a run-out at $Y = y$. Since there are no closed forms of the marginal pdf and of the marginal cdf of Y , they have to be evaluated numerically.

3.3.2 ML estimation of model parameters

The sample Likelihood for the CA S-N model is the resulting product of the marginal pdfs at failure data points and marginal cdfs at run-out data points. Considering the experimental data-set $(x_i, y_i)_{i=1, \dots, n_{tot}}$, the sample Likelihood is defined as follows:

$$L(\underline{\theta}) = \prod_{i=1}^{n_{tot}} (f_Y(\underline{\theta}; y_i, x_i))^{\delta_i} \cdot (1 - F_Y(\underline{\theta}; y_i, x_i))^{1-\delta_i} \quad \text{for } \delta_i = \begin{cases} 0 & \text{if } y_i \text{ is a run-out} \\ 1 & \text{if } y_i \text{ is a failure} \end{cases} \quad (\text{Eq. 24})$$

The sample Likelihood, $L(\underline{\theta})$, expresses the probability of observing $(y_1, \dots, y_{n_{tot}})$, at logstress ranges $(\exp(x_1), \dots, \exp(x_{n_{tot}}))$ for a given model parameter vector, $\underline{\theta}$. The ML estimate $\hat{\underline{\theta}}$ of $\underline{\theta}$ is the model parameter vector which maximizes the sample Likelihood, $L(\underline{\theta})$.

Generally it is easier to work with the negative sample log-Likelihood:

$$nLL(\underline{\theta}) = - \sum_{i=1}^{n_f} \ln(f_Y(\underline{\theta}; y_i, x_i)) - \sum_{i=1}^{n_r} \ln(1 - F_Y(\underline{\theta}; y_i, x_i)) \quad \text{with } \begin{cases} n_f : \text{nb. of failures} \\ n_r : \text{nb. of run-outs} \end{cases} \quad (\text{Eq. 25})$$

Then the ML estimate $\hat{\underline{\theta}}$ of $\underline{\theta}$ is the model parameter vector which minimizes the negative sample log-Likelihood, $nLL(\underline{\theta})$.

3.3.3 MCS estimation of p-quantile S-N curve

Under some regularity conditions, ML estimators have asymptotic normality property [57]: as the sample size increases, the distribution of ML estimators tends to the Normal distribution with mean equal to the true parameter vector being estimated and covariance matrix equal to the inverse of the observed Fisher Information matrix, $\underline{\underline{I}}(\hat{\underline{\theta}})$.

The Fisher information matrix, $\underline{\underline{I}}(\hat{\underline{\theta}})$, is the symmetrical matrix which contains following entries:

$$\underline{\underline{I}}(\underline{\theta}) = - \frac{\partial^2}{\partial \theta_i \partial \theta_j} L(\underline{\theta}) \quad (\text{Eq. 26})$$

The observed Fisher information matrix⁶, $\underline{\underline{I}}(\hat{\underline{\theta}})$, is the Fisher information matrix evaluated at the ML estimate, $\hat{\underline{\theta}}$.

The estimator of the asymptotic covariance matrix is then:

$$\underline{\underline{\rho}} = \underline{\underline{\text{Cov}}}(\underline{\theta}) = \left[\underline{\underline{I}}(\hat{\underline{\theta}}) \right]^{-1} \quad (\text{Eq. 27})$$

Hence under the hypothesis of asymptotic normality of ML estimators, the model parameter vector, $\underline{\theta}$, is multivariate Normal with mean equal to the ML estimate, $\hat{\underline{\theta}}$, and covariance matrix equal to the inverse of the observed Fisher information matrix (Equation 27).

In this work the following MCS scheme was designed in order to compute the p -quantile, $y_p(\tilde{x})$, of the fatigue log-life, at the log-stress range, $\exp(\tilde{x})$:

1. The p -value is chosen, with $P_f^t = p$.
2. The parameter η_{sam} (sample size, typically $1 \cdot 10^5$) is chosen.
3. The sample $\underline{\theta}_{i=1, \dots, \eta_{sam}}$ is generated from the multivariate normal distribution $\mathcal{N}(\hat{\underline{\theta}}, \underline{\underline{\rho}})$; then, $y(\underline{\theta}_i, \tilde{x})_{i=1, \dots, \eta_{sam}}$, $v(\underline{\theta}_i)_{i=1, \dots, \eta_{sam}}$, are sampled by using the sampled $\underline{\theta}_{i=1, \dots, \eta_{sam}}$. The epistemic uncertainty of S-N model parameters is taken into account in the $\underline{\theta}_{i=1, \dots, \eta_{sam}}$ sampling, while the aleatory uncertainty of the CAFL and the aleatory uncertainty of the fatigue life are taken into account in the $y(\underline{\theta}_i, \tilde{x})_{i=1, \dots, \eta_{sam}}$, $v(\underline{\theta}_i)_{i=1, \dots, \eta_{sam}}$ sampling.
4. For each element of the sample, the probability $P_{f_i}(y_p(\tilde{x}))$ is computed:

$$P_{f_i}(y_p(\tilde{x})) = \begin{cases} 0 & \text{if } y(\underline{\theta}_i, \tilde{x}) \geq y_p(\tilde{x}) \quad \text{or} \quad v(\underline{\theta}_i) \geq \tilde{x} \\ 1 & \text{if } y(\underline{\theta}_i, \tilde{x}) < y_p(\tilde{x}) \quad \text{and} \quad v(\underline{\theta}_i) < \tilde{x} \end{cases}$$

5. Objective function, $\mathcal{O}(y_p(\tilde{x}))$, is built:

$$\mathcal{O}(y_p(\tilde{x})) = \left[\frac{(\sum_{i=1}^{\eta_{sam}} P_{f_i}(y_p(\tilde{x})))}{\eta_{sam}} - P_f^t \right]^2$$

6. p -quantile of the fatigue log-life, at stress range $\exp(\tilde{x})$, is computed by minimizing the objective function, $\mathcal{O}(y_p(\tilde{x}))$:

$$\hat{y}_p(\tilde{x}) = \arg \min_{y_p(\tilde{x})} (\mathcal{O}(y_p(\tilde{x})))$$

⁶ The observed Fisher information matrix is computed in **CAestimation** module of Matlab Toolbox TB1 by using the built-in function **hessian(f,v)** where f is the scalar likelihood function and v is the ML estimate of model parameter vector.

The 0.05 quantile S-N curve, obtained with the scheme above, represents ML-based characteristic CA S-N curve. Since ML-based characteristic S-N curve is non-linear (see Fig. 6), a linearization scheme is proposed here for direct comparison with characteristic S-N curves from standards:

1. A straight line with slope equal to $\hat{m}_1$ (ML estimate of m_1 parameter) and passing through the true 0.05 quantile of the fatigue life at the maximum tested stress range, is drawn;
2. The line above is intersected with the horizontal line representing the true 0.05 quantile of the CAFL distribution.

The ML-MCS-based linearized characteristic S-N curve is shown in Fig. 6.

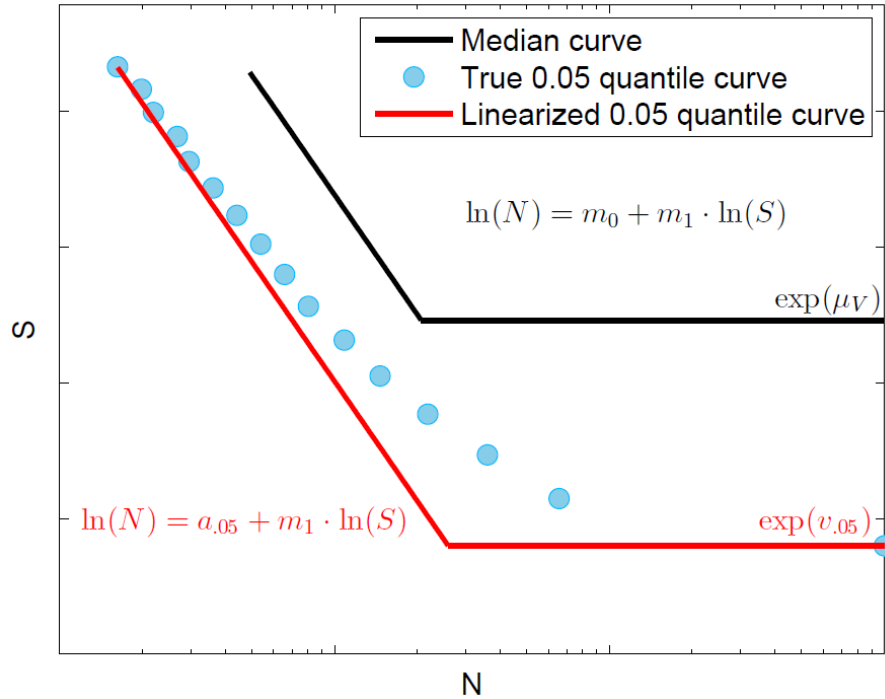


Fig. 6 ML-MCS-based median, 0.05 quantile and linearized 0.05 quantile S-N curves.

3.3.4 Mean stress effect

In the presented ML-MCS approach the effect of mean stress on fatigue strength is not explicitly taken into account by S-N model parameters (see Equation 14). The motivation for this choice is given below.

In welded joints, tensile residual stresses are introduced during the cooling process after the welding; these tensile stresses increase the mean stress and they can shorten the associated fatigue life [48]. Under the fatigue loading, a big part of residual stresses quickly relaxes (usually at first application of the maximum stress) [14],[70],[44], although the remaining part is still significant until the end of fatigue life [14],[54]. CA fatigue tests on longitudinal and transverse attachments from Maddox [50], Gurney [35],[36], Sonsino et al. [76], Röhrup et al. [69],[68] and Polezhayeva [65] have shown that in as-welded specimens, the compressive part of stress range cycles with stress ratio $-1 \leq R \leq 0$ does not have significant beneficial effect on associated fatigue life, due to the unrelaxed tensile residual stresses.

These results supported the choice of not explicitly including the mean stress effect into the S-N stochastic model; minor additional scatter in fatigue life, caused by the above discussed means stress effect, is taken into account by the model error term, $\varepsilon(0, \exp(\sigma))$.

For $R \rightarrow -\infty$ (fully compressive cycles) the beneficial effect of the compressive part of stress range cycles is significant [50],[68],[69],[65] and should be taken into account in the formulation of the S-N model; this case was not considered in this study.

3.4 Study cases

The CA S-N stochastic model was fitted to two different fatigue data-sets:

1. From fatigue testing of end-welded cover plate beams, under CA loading (see Section 3.4.1);
2. From fatigue testing of welded in-plane gussets, under CA loading (see Section 3.4.2).

These data-sets are reported, with some differences/errors, in the Commentary to Eurocode 3, part 1-9 [74].

3.4.1 Cover plate

CA experimental data-set for cover plate detail is made of 26 data points from testing of endwelded cover plate beams at CA stress ranges between 27.6 and 55.2MPa [29] (see Tab. 1). The thickness of the beam flange, t_f , is 9.78 mm, while the thickness of the coverplate, t_c ranges between 13.1mm and 14.3mm (see Fig. 7); the detail is classified as FAT50 (with CAFL=37MPa) according to Eurocode EN1993-1-9 [2] and IIW recommendations [42] (with CAFL=29 MPa), and as FAT 56 (with CAFL=31 MPa) according to AASHTO bridge design specifications [4].

Cover plate with size effect correction

Since the detail considered for CA experimental data-set (FAT 50 according to EN 1993-1-9) has not the same classification as the detail considered for VA experimental data-set (see Section 4.3, $t_f = t_c = 25.4 \text{ mm}$, FAT 45 according to EN 1993-1-9), the observed stress ranges, S_i , are reduced by a factor ($45/50=0.9$) for size effect to be consistent with the VA experimental data-set detail classification. It is noted that the detail is classified as FAT 50 (with CAFL=29MPa) according to IIW recommendations [42], where the influence of t_c and t_f is not taken into account, and as FAT 40 (with CAFL=18 MPa) according to AASHTO bridge design specifications [4], where the influence of t_c and t_f is stronger than in Eurocode EN1993-1-9 [2].

3.4.2 In-plane gusset

CA experimental data-set for in-plane gusset (see Tab. 2 and Fig. 8) is made of:

- 6 data points from testing of in-plane welded gussets with attachment length, $L=100\text{mm}$ at CA stress ranges between 80 and 160MPa [41];
- 1 data points from testing of in-plane welded gussets with attachment length, $L=150\text{mm}$ at CA stress range equal to 50MPa (ICOM tests, [10]);
- 10 data points from testing of in-plane welded gussets with attachment length, $L=100\text{mm}$ at CA stress ranges between 55 and 140MPa [47];
- 12 data points from testing of in-plane welded gussets with attachment length, $L=100\text{mm}$ at CA stress ranges between 33 and 100MPa [8].

For $L \leq 150\text{mm}$, the detail is classified as FAT 40 (with CAFL=31 MPa) according to EN 1993-1-9 [74], as FAT50 (with CAFL=29MPa) according to IIW recommendations [42], and as FAT56 (with CAFL=31MPa) according to AASHTO bridge design specifications [4].

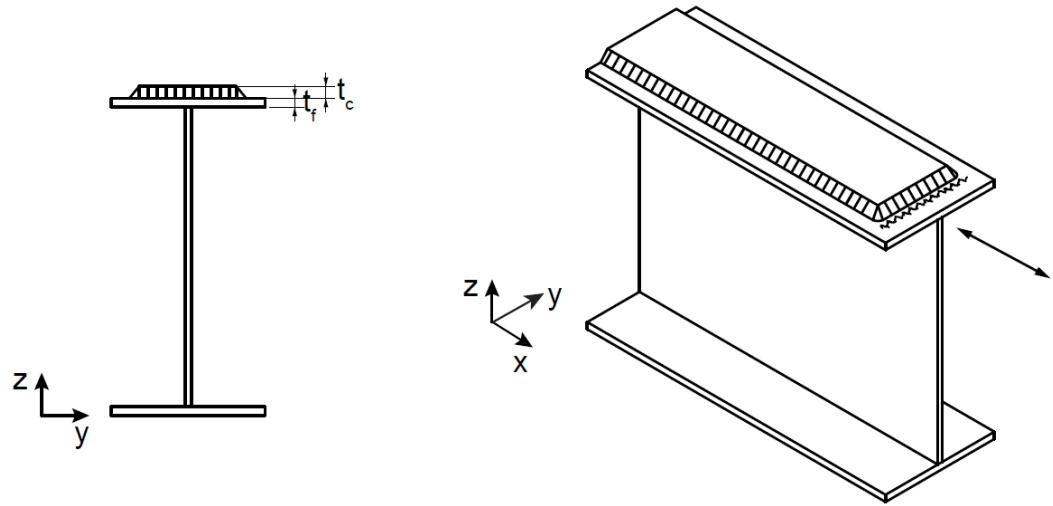


Fig. 7 Cover plate detail.

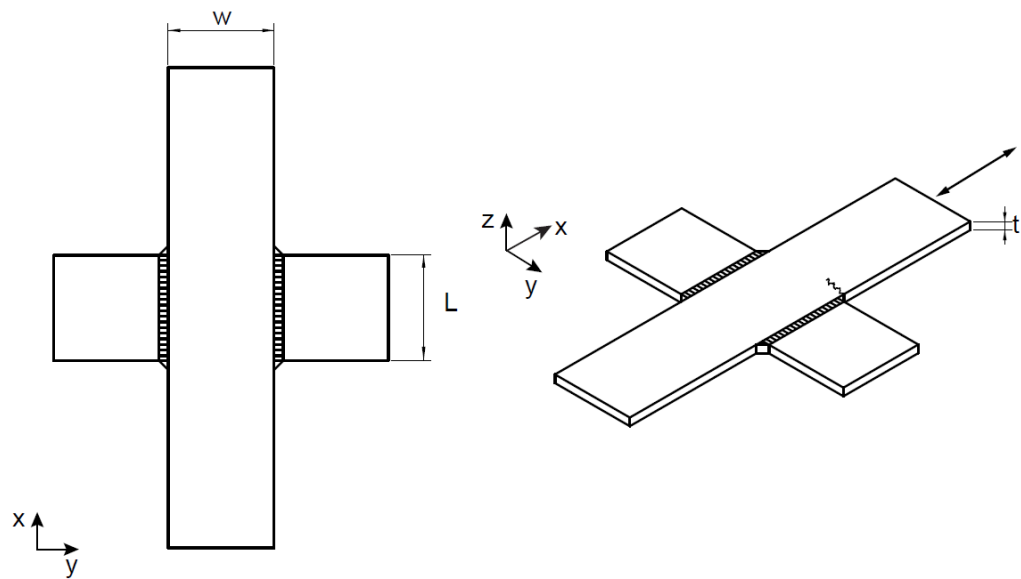


Fig. 8 In-plate gusset detail.

Tab. 1 Cover plate, CA experimental data-set. δ_i is a binary variable which is equal to 1 for a failure and to 0 for a run-out

Test series	S [MPa]	N	δ_i	Failure location
Fisher (1977)	27.6	100000000	0	-
	27.6	100000000	0	-
	27.6	100000000	0	-
	27.6	100000000	0	-
	31.7	100000000	0	-
	31.7	100000000	0	-
	31.7	100000000	0	-
	31.7	100000000	0	-
	32.4	34930000	1	weld toe
	32.4	37714000	1	weld toe
	34.5	16613000	1	weld toe
	34.5	32505000	1	weld toe
	34.5	8451000	1	weld toe
	34.5	47293000	1	weld toe
	34.5	89314000	0	-
	34.5	89314999	0	-
	41.4	11418000	1	weld toe
	41.4	12158000	1	weld toe
	41.4	4327000	1	weld toe
	41.4	12158000	1	weld toe
	41.4	100000000	0	-
	41.4	100000000	0	-
	55.2	2334000	1	weld toe
	55.2	5006000	1	weld toe
	55.2	4235000	1	weld toe
	55.2	1351300	1	weld toe

Tab. 2 In-plane gusset, CA experimental data-set. δ_i is a binary variable which is equal to 1 for a failure and to 0 for a run-out

Test series	S [MPa]	N	δ_i	Failure location
Hirt (1975) L=100mm	80	1922000	1	weld toe
	80	1810000	1	weld toe
	120	514000	1	weld toe
	120	361000	1	weld toe
	160	210000	1	weld toe
	160	199000	1	weld toe
ICOM (2015), L=150mm	50	3800000	1	weld toe
Kondo (2002) L=100mm	140	528000	1	weld toe
	140	252000	1	weld toe
	140	272000	1	weld toe
	100	836000	1	weld toe
	80	1409000	1	weld toe
	80	1431000	1	weld toe
	60	21620000	0	-
	60	5492000	1	weld toe
	60	5144000	1	weld toe
	55	4049000	1	weld toe
Bae (2004) L=150mm	100	387000	1	weld toe
	100	561000	1	weld toe
	100	721000	1	weld toe
	100	787000	1	weld toe
	66	1000000	1	weld toe
	66	1430000	1	weld toe
	66	1710000	1	weld toe
	66	1730000	1	weld toe
	33	13430000	0	-
	33	16600000	0	-
	33	18630000	0	-
	33	23420000	0	-

3.5 Results

The ML estimate and the standard deviation of the CA S-N model vector parameters are shown in:

- Tab. 3 for the cover plate data-set;
- Tab. 4 for the cover plate data-set with size effect correction;
- Tab. 5 for the in-plane gusset data-set.

The value of the negative log-Likelihood at the ML estimate is also shown in the Tables above.

In all three data-sets the case $(V = \mathcal{N})$ gives a better fit in terms of sample log-Likelihood, with respect to the case $(V = EV)$. The difference in the sample log-Likelihood is trivial but in any case does not support the case $(V = EV)$ to the detriment of the conventional case $(V = \mathcal{N})$. The size effect correction in the cover plate data-set has small influence on ML estimate of the intercept of the S-N curve, $\hat{m}_0$ (which drops from 29.02 to 28.66, for the case $V = \mathcal{N}$), while it has quite significant influence on ML estimate of the location parameter of the log-CAFL distribution, μ_V (which drops from 3.56 to 3.45, for the case $V = \mathcal{N}$).

The correlation coefficients $c_{ij} = \frac{\rho_{ij}}{\sqrt{\rho_{ii}\rho_{jj}}}$ of the CA S-N model are shown in:

- Fig. 9 for the cover plate data-set;
- Fig. 10 for the cover plate data-set with size effect correction;
- Fig. 11 for the in-plane gusset data-set.

where the covariance coefficients ρ_{ij} are calculated by inverting the observed Fisher matrix, as explained in Section 3.3.3.

For all three data-sets, the correlation coefficient $c_{m_0 m_1}$ is equal to -1.00, which indicates a perfect negative linear dependency between the intercept of the S-N curve, m_0 , and the slope of the S-N curve, m_1 . For the cover plate data-set and for the cover plate dataset with size effect correction, the correlation coefficient $c_{\mu_V \sigma_V}$ is equal to -0.03, which indicates that there is no linear correlation between the location parameter and the scale parameter of the log-CAFL probability distribution. For the in-plane gusset data-set, the correlation coefficient $c_{\mu_V \sigma_V}$ is equal to -0.62, which indicates a moderate negative linear dependency between the location parameter and the scale parameter of the log-CAFL distribution. For all three data-sets, there is no linear correlation between: 1) The S-N curve variance parameter, σ , and all other parameters; and 2) The S-N curve parameters m_0 , m_1 and the log-CAFL parameters μ_V , σ_V .

Tab. 3 Cover plate, CA S-N model parameters

Parameter	ML estimate $V = \mathcal{N}$		ML estimate $V = EV$	
	MLE	St.Dev.	MLE	St.Dev.
m_0	20.017	2.935	29.138	3.002
m_1	-3.416	0.786	-3.448	0.804
σ	-0.544	0.197	-0.540	0.199
μ_V	3.556	0.0577	3.646	0.0679
σ_V	-1.623	0.408	-1.592	0.440
$n\hat{L}L$	24.47		24.74	

$$\underline{\underline{c}} = \begin{bmatrix} 1.00 & -1.00 & 0.07 & -0.05 & 0.06 \\ \vdots & 1.00 & -0.07 & 0.05 & -0.06 \\ \vdots & \vdots & 1.00 & -0.04 & 0.04 \\ \vdots & \vdots & \vdots & 1.00 & -0.03 \\ \vdots & \vdots & \vdots & \vdots & 1.00 \end{bmatrix}$$

Fig. 9 Cover plate, CA S-N model correlation matrix, case ($V = \mathcal{N}$).**Tab. 4** Cover plate with size effect correction, CA S-N model parameters

Parameter	ML estimate $V = \mathcal{N}$		ML estimate $V = EV$	
	MLE	St.Dev.	MLE	St.Dev.
m_0	28.657	2.852	28.775	2.920
m_1	-3.416	0.786	-3.448	0.804
σ	-0.544	0.197	-0.540	0.199
μ_V	3.451	0.0577	3.540	0.0679
σ_V	-1.623	0.408	-1.592	0.440
$n\hat{L}L$	24.47		24.74	

$$\underline{\underline{c}} = \begin{bmatrix} 1.00 & -1.00 & 0.07 & -0.05 & 0.06 \\ \vdots & 1.00 & -0.07 & 0.05 & -0.06 \\ \vdots & \vdots & 1.00 & -0.04 & 0.04 \\ \vdots & \vdots & \vdots & 1.00 & -0.03 \\ \vdots & \vdots & \vdots & \vdots & 1.00 \end{bmatrix}$$

Fig. 10 Cover plate with size effect correction, CA S-N model correlation matrix, case ($V = \mathcal{N}$).

Tab. 5 In-plane gusset, CA S-N model parameters

Parameter	ML estimate $V = \mathcal{N}$		ML estimate $V = EV$	
	MLE	St.Dev.	MLE	St.Dev.
m_0	25.770	0.945	25.804	0.954
m_1	-2.666	0.209	-2.674	0.211
σ	-1.048	0.144	-1.048	0.144
μ_V	3.864	0.127	3.966	0.104
σ_V	-1.667	0.498	-1.712	0.590
$n\hat{L}L$	12.34		13.52	

$$\underline{\underline{c}} = \begin{bmatrix} 1.00 & -1.00 & 0.00 & -0.06 & 0.06 \\ \vdots & 1.00 & -0.00 & 0.05 & -0.06 \\ \vdots & \vdots & 1.00 & -0.00 & 0.00 \\ \vdots & \vdots & \vdots & 1.00 & -0.62 \\ \vdots & \vdots & \vdots & \vdots & 1.00 \end{bmatrix}$$

Fig. 11 In-plane gusset, CA S-N model correlation matrix, case ($V = \mathcal{N}$).

3.5.1 S-N curves – Cover plate

Fig. 12 shows the ML-MCS-based median S-N curve and ML-MCS-based 0.05 quantile linearized S-N curve, for the cover plate data-set. The non-linear 0.05 quantile S-N curve was computed using the MCS scheme presented in Section 3.3.3, with $\eta_{sam} = 1 \cdot 10^5$; the non-linear 0.05 quantile S-N curve was then linearized as explained in Section 4.3.3: the detail is classified as FAT51, with CAFL=24MPa and knee point at $27 \cdot 10^6$ cycles. Fig. 14 shows the median S-N curve and the characteristic S-N curve based on the statistical method recommended in Commentary to Eurocode 3, part 1-9 [74] (see Section 3.2), for the cover plate data-set: the detail is classified as FAT 47 with CAFL=36MPa assumed to

start at $5 \cdot 10^6$ cycles⁷. The comparison of the ML-MCS-based 0.05 quantile linearized S-N curve with the characteristic S-N curves from standards, for the cover plate data-set (see Fig. 16), gives the following information:

- The Commentary to Eurocode 3-based S-N curve and the Eurocode EN 1993-1-9 FAT 50 S-N curve are almost coincident;
- The estimation of the fatigue strength in the finite life (FL) region ($N < 5 \cdot 10^6$), according to EN 1993-1-9 FAT 50 and the IIW FAT 50 S-N curves, is slightly over-conservative with respect to the ML-MCS S-N curve: the probability of failure is slightly lower than 5%;
- The estimation of the fatigue strength in the FL region ($N < 5 \cdot 10^6$), according to AASHTO FAT 56 S-N curve, is slightly under-conservative with respect to the ML-MCS S-N curve: the probability of failure is slightly higher than 5%;
- The estimation of the CAFL, according to EN3 FAT 50 S-N curve, is highly unsafe: 6 failures point lie below the EN3-based CAFL and the probability of exceeding the EN3-based CAFL is considerably lower than 95%;
- The estimation of the CAFL, according to IIW FAT 50 and the AASHTO FAT 56 S-N curves, is slightly under-conservative with respect to the ML-MCS S-N curve: the probability of exceeding the IIW-based and the AASHTO-based CAFL is lower than 95%.

Fig. 13 shows the ML-MCS-based median S-N curve and the ML-MCS-based 0.05 quantile linearized S-N curve, for the cover plate data-set with size effect correction. Due to the reduction of the nominal applied stress ranges, the detail drops from FAT51 to FAT46 and the CAFL drops from 24MPa to 21MPa; the position of the knee point is not affected by the reduction of the nominal applied stress ranges. The same reduction is observed in Commentary to Eurocode 3 document-based characteristic S-N curve (see Fig. 15). The comparison of the ML-MCS-based 0.05 quantile linearized S-N curves with the characteristic S-N curves from standards, for the cover plate data-set with size effect correction (see Fig. 17), gives the following information:

- The Commentary to Eurocode 3 document-based and the Eurocode EN1993-1-9 FAT50 characteristic S-N curves are almost coincident;
- The estimation of the fatigue strength in the FL region ($N < 5 \cdot 10^6$), according to EN3 FAT 45 S-N curves, is slightly over-conservative with respect to the ML-MCS S-N curve: the probability of failure is slightly lower than 5%;
- The estimation of the fatigue strength in the FL region ($N < 5 \cdot 10^6$), according to IIW FAT 50 characteristic S-N curves, is slightly under-conservative with respect to the ML-MCS S-N curve: the probability of failure is slightly higher than 5%;
- The estimation of the fatigue strength in the FL region ($N < 5 \cdot 10^6$), according to AASHTO FAT 40 S-N curve, is considerably over-conservative with respect to the ML-MCS S-N curve: the probability of failure is considerably lower than 5%;
- The estimation of the CAFL, according to EN3 FAT 45 S-N curve, is highly unsafe: 6 failures point lie below the EN3-based CAFL and the probability of exceeding the EN3-based CAFL is considerably lower than 95%;
- The estimation of the CAFL, according to IIW FAT 50 S-N curve, is slightly unsafe: 2 failures point lie below the EN3-based CAFL and the probability of exceeding the IIW-based CAFL is slightly lower than 95%;
- The estimation of the CAFL, according to AASHTO FAT40 S-N curve, is over-conservative with respect to the ML-MCS S-N curve: the probability of exceeding the AASHTO-based CAFL is higher than 95%.

⁷ The difference between the S-N curve based on Commentary to Eurocode 3 and the S-N curve based on Eurocode 3 is that the first one is obtained by applying the Eurocode statistical approach to the experimental data-sets considered in this study, while the latter one is the S-N curve corresponding to the FAT classification of the considered detail and which is not related to the experimental data-set considered in this study.

3.5.2 S-N curves – In-plane gusset

Fig. 18 shows the ML-MCS-based median S-N curve and ML-MCS-based 0.05 quantile linearized S-N curve, for the in-plane gusset data-set: the detail is classified as FAT 54, with CAFL=30MPa and knee point at $9.8 \cdot 10^6$ cycles. Fig. 19 shows the median S-N curve and the characteristic S-N curve based on the statistic method recommended in EN3 Commentary document (see Section 3.2), for the in-plane gusset data-set: the detail is classified as FAT 53 with CAFL=38 MPa assumed to start at $5 \cdot 10^6$ cycles. The comparison of the ML-MCS-based 0.05 quantile linearized S-N curve with the characteristic S-N curves from standards, for the in-plane gusset data-set (see Fig. 20), gives the following information:

- The EN3 FAT 40 S-N curve is more conservative than the EN3 Commentary doc.-based S-N curve (FAT53);
- The IIW FAT 50 S-N curve, the AASHTO FAT 56 S-N curve and the ML-MCS-based S-N curve are almost coincident in the FL region ($N < 5 \cdot 10^6$ cycles);
- The estimation of the fatigue strength in the FL region ($N < 5 \cdot 10^6$), according to EN3 FAT40 S-N curve, is over-conservative with respect to the ML-MCS-based S-N curve: the probability of failure is lower than 5%;
- The AASHTO FAT56 S-N curve, the IIW FAT50 S-N curve and ML-MCS-based S-N curve, give very similar estimation of the CAFL;
- The estimation of the CAFL, according to EN3 Commentary doc.-based S-N curves, is under-conservative with respect to the ML-MCS S-N curve, however no failure points exist to confirm this.

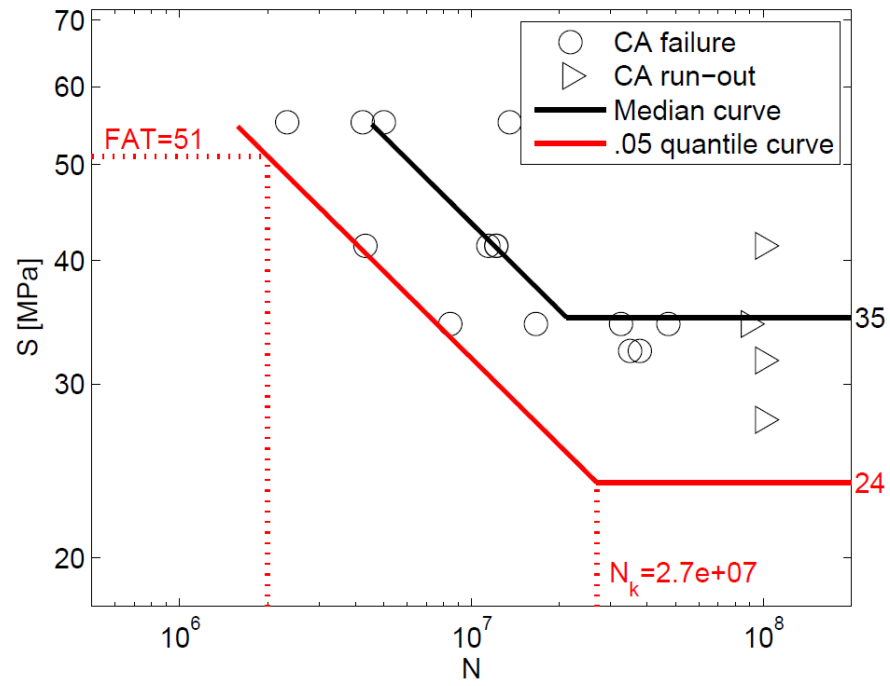


Fig. 12 Cover plate, ML-MCS-based S - N curves.

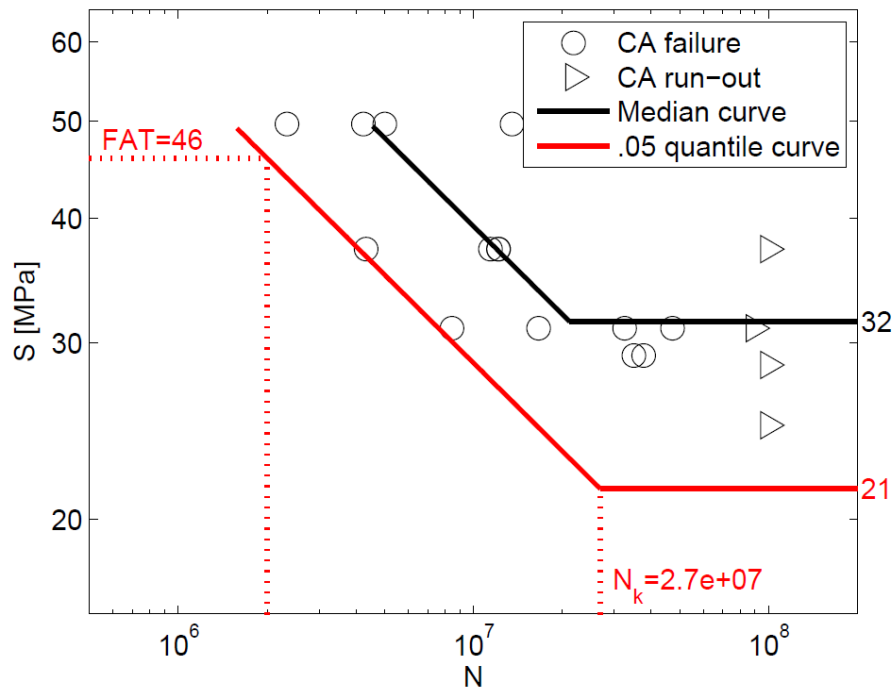


Fig. 13 Cover plate with size effect correction, ML-MCS-based S - N curves.

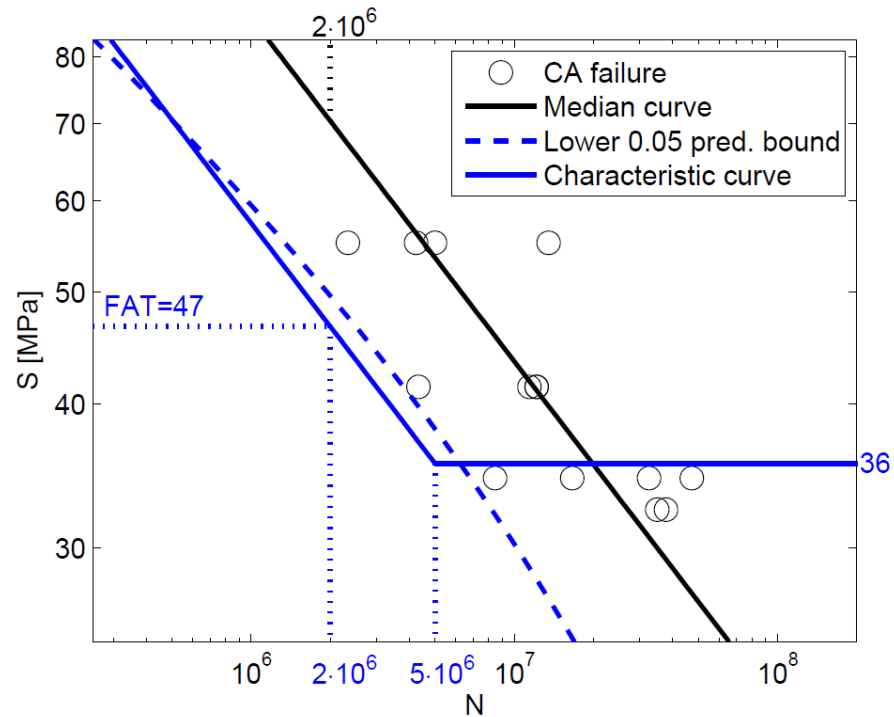


Fig. 14 Cover plate, Comment. to Eurocode3-based S-N curves.

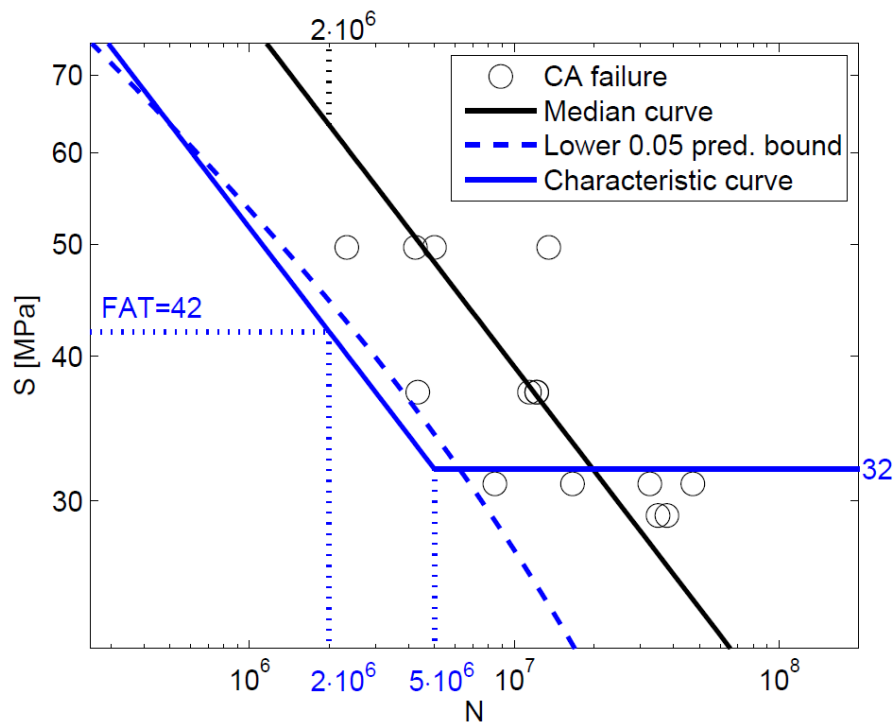


Fig. 15 Cover plate with size effect correction, Comment. to Eurocode3-based S-N curves.

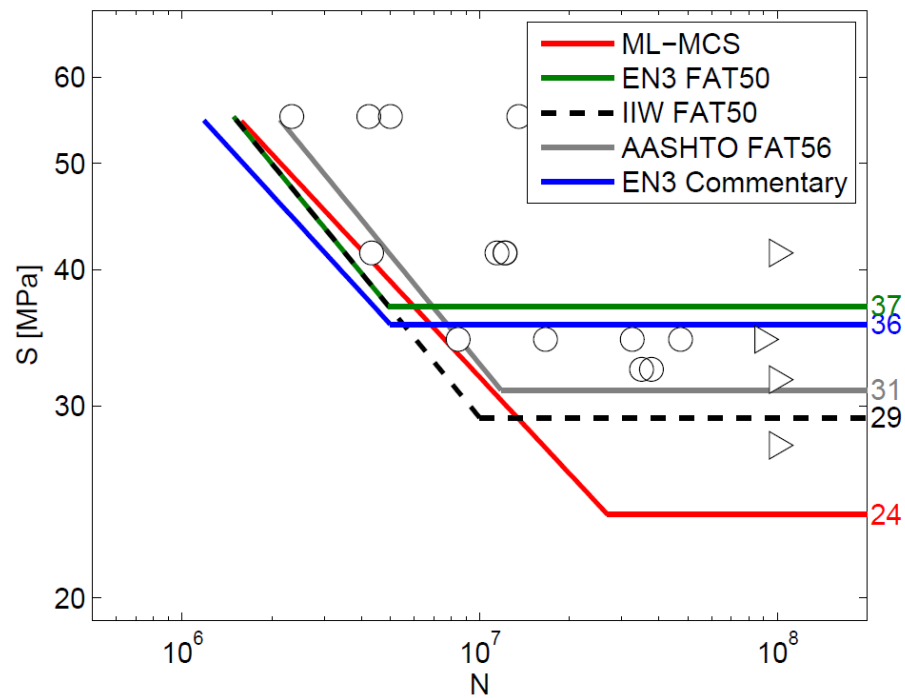


Fig. 16 Cover plate, characteristic S-N curves.

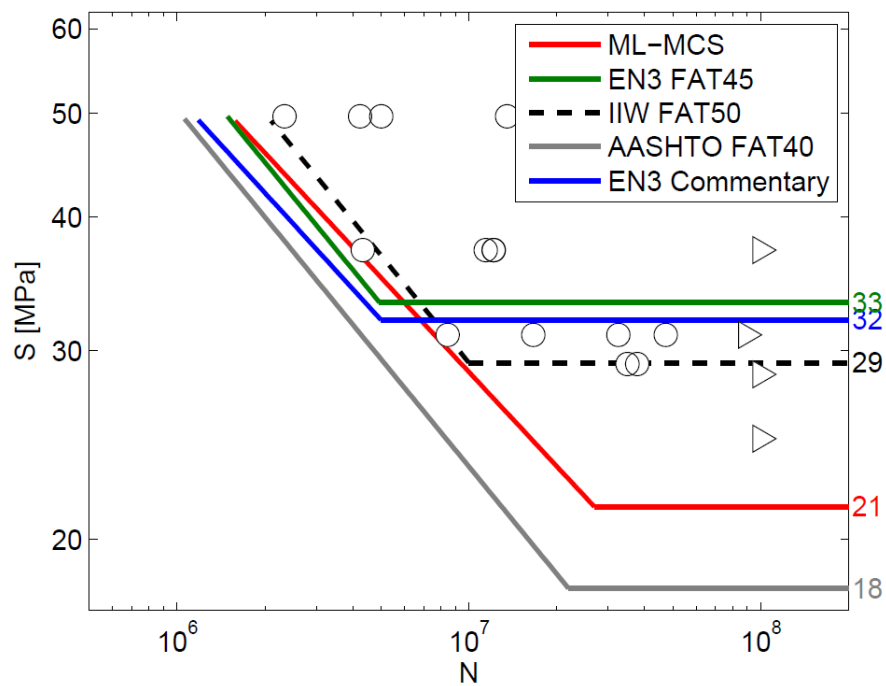


Fig. 17 Cover plate with size effect correction, characteristic S-N curves.

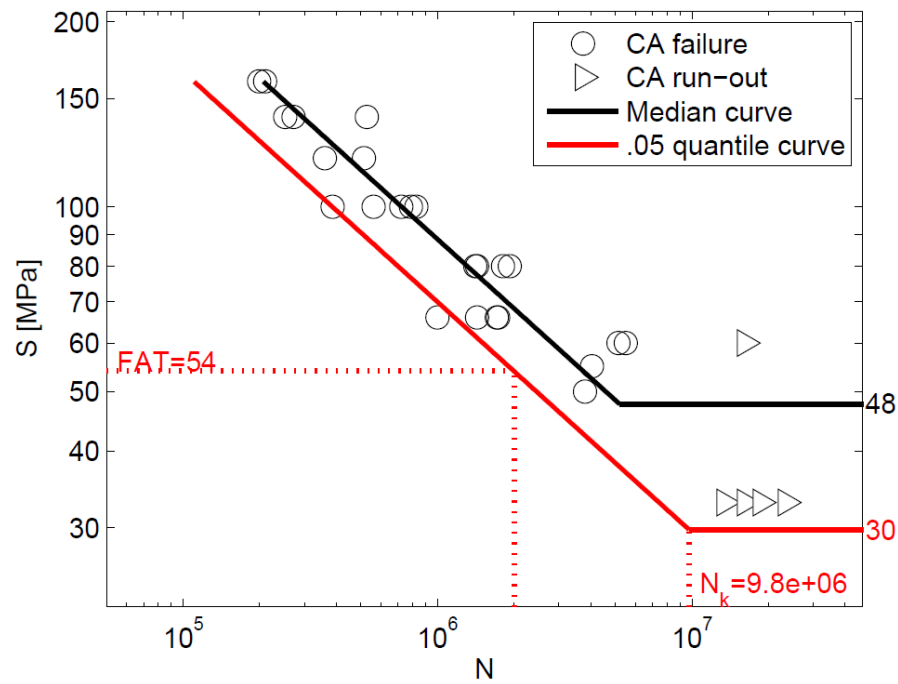


Fig. 18 In-plane gusset, ML-MCS-based S-N curves.

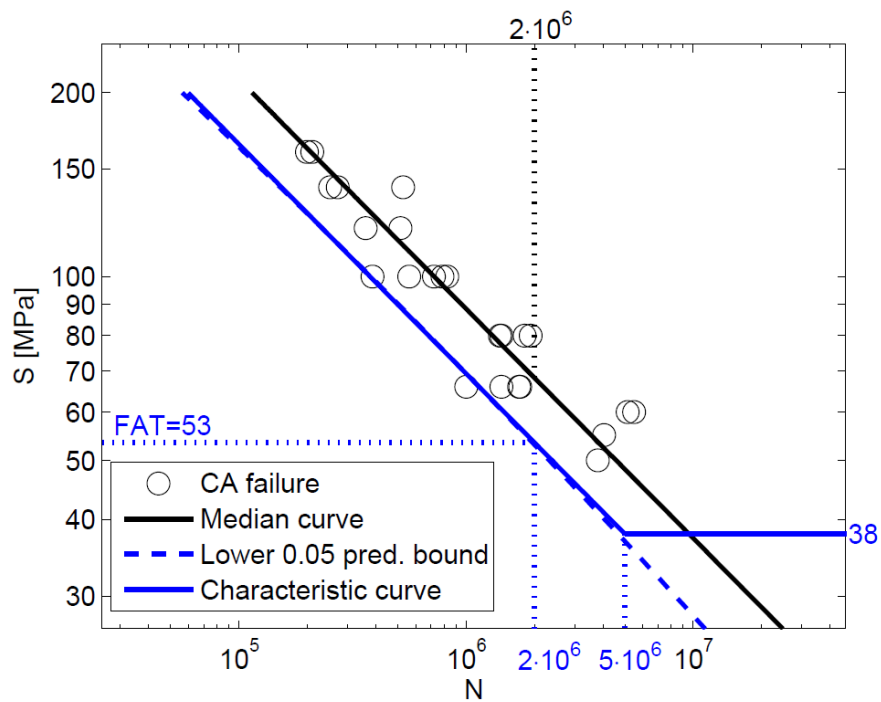


Fig. 19 In-plane gusset, Comment. to Eurocode3-based S-N curves.

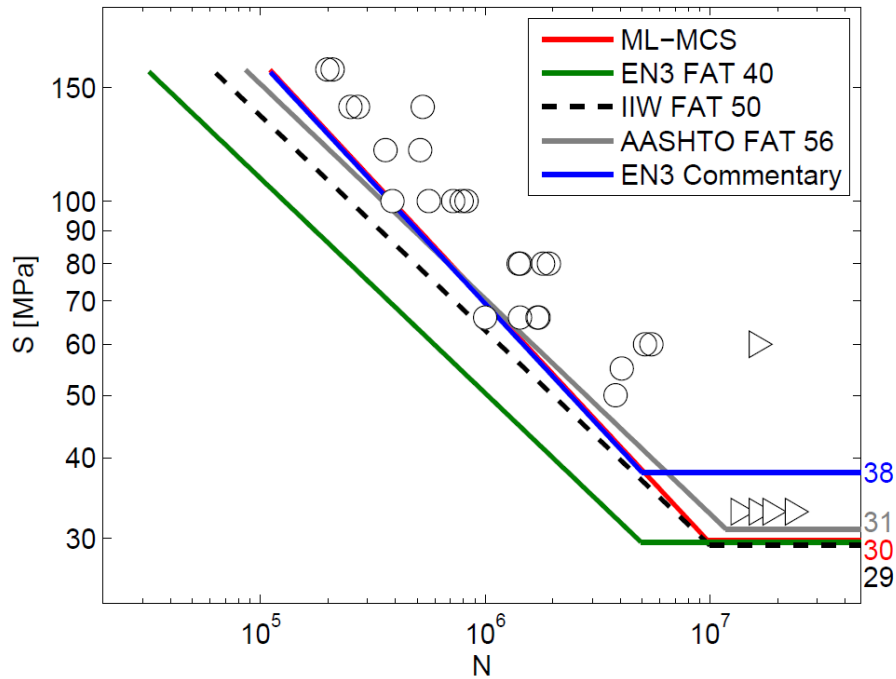


Fig. 20 In-plane gusset, characteristic S - N curves.

It is remarked that the ML-MCS scheme used for estimation of characteristic S - N curves takes into account both: 1) The epistemic uncertainty of S - N model parameters; and 2) The aleatory uncertainty of the CAFL and of the fatigue life.

In order to assess the influence of the aleatory uncertainty following analysis was carried out, by considering the cover-plate and the in-plane gusset data-sets:

1. 10^6 S - N curves were sampled by only considering the epistemic uncertainty of S - N model parameters; two vectors of size $(10^6 \times 1)$ were created by taking from each sampled curve the CAFL and the number of cycle N^* corresponding to the fatigue strength at $2 \cdot 10^6$ cycles according to the median S - N curve ($S = 69.9$ MPa for the cover-plate, while $S = 68.3$ MPa for the in-plane gusset). These vectors were fitted with two log-Normal distributions (CAFL and N^* log-Normal distributions);
2. Step 1 was repeated by considering both the epistemic uncertainty of S - N model parameters and the aleatory uncertainty of the fatigue life and of the CAFL;
3. Log-Normal distributions of the CAFL and of N^* estimated at Step 1 were compared to log-Normal distributions of the CAFL and of N^* estimated at Step 2, in terms of the coefficient of variation.

For the cover-plate, Fig. 21 shows that: 1) The coefficient of variation of the $N^* = N(S = 69.9 \text{ MPa})$ distribution significantly increases from 0.46 to 0.86, by taking into account both the epistemic uncertainty of S - N model parameters and the aleatory uncertainty of the fatigue life, with respect to the case where only the epistemic uncertainty of S - N model parameters is considered; and 2) The coefficient of variation of the CAFL distribution notably increases from 0.06 to 0.24, by taking into account both the epistemic uncertainty of S - N model parameters and the aleatory uncertainty of the CAFL, with respect to the case where only the epistemic uncertainty of S - N model parameters is considered.

For the in-plane gusset, Fig. 22 confirms the strong influence of the aleatory uncertainty of the fatigue life and of the aleatory uncertainty of the CAFL, respectively on the $N^* = N(S = 68.3 \text{ MPa})$ distribution (the coefficient of variation increases from 0.09 to

0.38) and on the CAFL distribution (the coefficient of variation increases from 0.13 to 0.28).

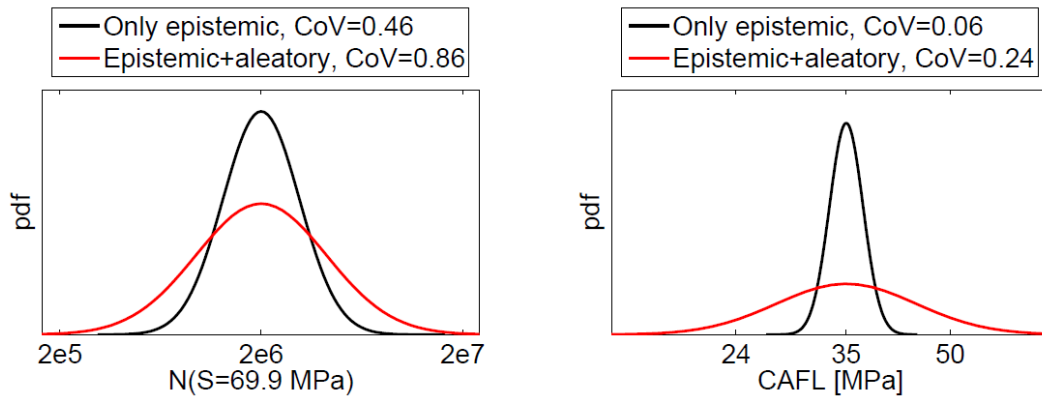


Fig. 21 Influence of the aleatory uncertainty of the fatigue life and of the CAFL, cover-plate.

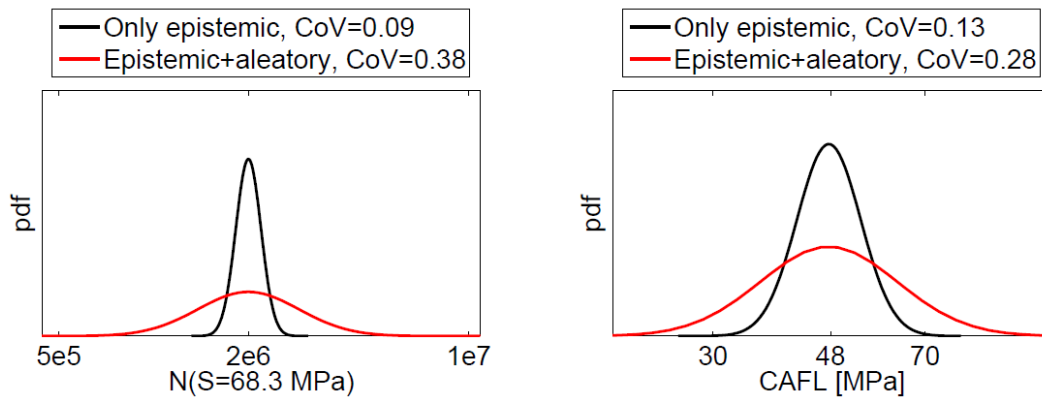


Fig. 22 Influence of the aleatory uncertainty of the fatigue life and of the CAFL, in-plane gusset.

3.6 Conclusions and discussion

In this Chapter a new approach for estimation of CA S-N curves of components was presented, with application to three study cases of welded joints. The new approach has general applicability to all types of details (i.e. non-welded and welded joints). It was implemented in the Matlab Toolbox TB1 (see Appendix III) allowing for ML fitting of linear S-N model to experimental fatigue data-sets and MCS estimation of true p-quantile S-N curves. The ML-MCS approach-based characteristic S-N curves were linearized for direct comparison with current standards.

The conclusions from this Chapter are as follows:

- The statistical method for evaluation of fatigue S-N curves recommended by Commentary to Eurocode 3, part 1-9 document [74] is affected by several limitations⁸: 1) The median S-N curve is estimated by fitting a linear regression to failure data and disregarding run-out data, resulting in loss of information; 2) The characteristic curve is estimated by assuming the fatigue data-set as homoscedastic and by considering the fatigue scatter at $2 \cdot 10^6$ cycles as representative of fatigue scatter at all number of cycles; and 3) The CAFL is arbitrarily assumed to start at $5 \cdot 10^5$ cycles. The ML-

⁸ The statistical procedures for evaluating S-N curves, recommended in other standards (IIW, AASHTO), show minor differences with respect to Eurocode recommendations and they are affected by the same limitations.

MCS approach presented here overcomes these limitations by using run-out data for ML estimation of S-N model, and by including the CAFL and the fatigue scatter in the stochastic model;

- The RFL model proposed by Pascual [63] allows to overcome limitations of Eurocode EN1993-1-9 approach but does not give straightforward method for estimating characteristic S-N curves; the ML-MCS approach presented here allows for computation of true p – quantiles of fatigue life and direct estimation of characteristic S-N curves. Moreover the RFL fitting model is not linear and direct comparison with current standards is not straightforward as for ML-MCS-based S-N curves;
- The estimates of CA S-N stochastic model for all considered data-sets show that the choice of modeling the CAFL as a log-Normal random variable maximizes the sample Likelihood information (see Tab. 3 and Tab. 5). In order to make the coverplate CA data-set consistent with the VA cover plate data-set (used for estimation of VA S-N stochastic model, see Section 4.3), a size effect correction was applied to the coverplate data-set; also for the cover plate data-set with size correction the choice of modeling the CAFL as a log-normal random variable maximizes the sample Likelihood information (see Tab. 4);
- The CA S-N model correlation matrix for all considered data-sets shows that there is a perfect negative linear correlation between the intercept and the slope of the S-N curve in the log-S-N plane, while it is realistic to assume that the parameters of the CAFL distribution are independent from the parameters of the S-N curve (see Fig. 9, Fig. 10 and Fig. 11);
- The ML-MCS characteristic S-N curves for all considered data-sets were linearized for direct comparison with current standards. Eurocode EN1993-1-9, AASHTO bridge design specifications and IIW recommendations provide estimation of fatigue strength in the FL region ($N < 5 \cdot 10^6$ cycles) which is similar to the estimation based on ML-MCS approach (see Fig. 16, Fig. 17 and Fig. 20).

A larger difference can be observed in the HCF region: for the cover plate dataset, all standards provide unsafe estimation of the CAFL and of the knee point (see Fig. 16); for the cover plate with size effect correction data-set both Eurocode and AASHTO-based S-N curves provide unsafe estimation of the CAFL and of the knee point (with significant amount of experimental failure points lying below the CAFL), while the AASHTO-based S-N curve provides over-conservative estimation of the CAFL (see Fig. 17); for the in-plane gussets all standards provide very similar estimation of CAFL (see Fig. 20);

- The assumption of having the knee point at a number of cycles which increases as the fatigue strength at 2 million cycles decreases (as assumed in AASHTO specifications) seems to be confirmed by estimated ML-MCS S-N curves (cover plate with size effect correction has FAT= 46 and knee point at $2.7 \cdot 10^7$ cycles, while the in-plane gusset has FAT= 54 and knee point at $9.8 \cdot 10^6$ cycles). This has to be confirmed by further experimental results.

The findings listed above suggest that the limitations included in the current standards could lead to unsafe estimation of fatigue strength, especially in the HCF region. The approach presented in this Chapter constitutes a powerful tool which can be used to redefine current standard S-N curves and relative FAT categories. It has to be noted that use of experimental fatigue data-sets containing significant information in the region between $5 \cdot 10^6$ and 10^8 cycles will reduce the uncertainty on estimation of the CAFL and of the knee point and it will improve the confidence in estimated S-N curves.

4 S-N curves for variable amplitude loadings

4.1 Introduction

Components, including welded joints, in road bridges are subjected to VA fatigue loading histories. The fatigue design of new bridges and the fatigue assessment of existing bridges are based on CA S-N curves, used in combination with Miner's rule. CA S-N curves are based on fatigue data from CA experimental tests. According to Miner's rule, following condition has to be verified for fatigue verification:

$$\sum_{i=1}^{n_{tot}} \frac{n_i}{N_i} \leq D_c \quad (\text{Eq. 28})$$

where:

- n_i is the number of cycles corresponding to the applied nominal stress range, S_i ;
- n_{tot} is the total number of stress range cycles;
- N_i is the number of cycles to failure corresponding to S_i ;
- D_c is the critical value of the damage sum.

The first critical aspect in using Miner's rule is how to consider the influence on the fatigue damage of the stress range cycles below the CAFL. In Eurocode EN1993-1-9 [2] and in IIW recommendations [42], the CA characteristic S-N curve is bent at the CAFL position from the slope $m_1 = -3$ to the slope $m_2 = -5$; a cut-off limit is fixed at 10^8 cycles. The use of the reduced (flatter) slope $m_2 = -5$ has been proposed by Haibach [37], by assuming a continuously decreasing of the fatigue limit as a function of the linear accumulated damage [38].

According to the Haibach's hypothesis, the standard format of Miner's rule verification (see Equation 28) is replaced by:

$$\sum_{i=1}^{n_{tot1}} \frac{n_i}{N_i} + \sum_{j=1}^{n_{tot2}} \frac{n_j}{N_j} \leq D_c \quad (\text{Eq. 29})$$

where:

- n_i is the number of cycles corresponding to the applied nominal stress range, S_i , with $S_i \geq \text{CAFL}$;
- n_j is the number of cycles corresponding to the applied nominal stress range, S_j , with $S_j < \text{CAFL}$;
- n_{tot1} and n_{tot2} are respectively the total number of stress range cycles above and below the CAFL;
- N_i is the number of cycles to failure corresponding to S_i ;
- N_j is the number of cycles to failure corresponding to S_j .

In AASHTO bridge design specifications [4] the CA S-N curve is extrapolated below the CAFL with no slope change, but a variable amplitude fatigue limit (VAFL) is set at CAFL/2, which is another way of dealing with all stress ranges lower than CAFL contribute less to fatigue damage.

Up to now, the use of the second slope m_2 has not been sufficiently justified by means of VA fatigue test results; at present the influence of the stress range cycles below the CAFL, on the fatigue damage, is still under debate.

Fisher et al. [28] performed fatigue tests of eight large beams with longitudinal attachments and welded cover plates, using different Rayleigh loading spectra with a CAFL⁹ exceedance rate ranging from 0.1% and 12%. They found that if any of the stress ranges in the loading spectrum exceeded the CAFL, then all cycles contribute to fatigue damage and the one-slope extrapolated S-N curve should be used.

Marquis [52] performed fatigue test of non-load carrying carbon and stainless steel fillet welds, at R=-1, using three different concave upwards truncated spectra (i.e. to simplify concave spectra according to the mathematical definition). He found that the use of the Miner's rule in combination with a bent S-N curve was unsafe, while the use of the extrapolated one-slope S-N curve gave conservative estimate of fatigue damage.

Gurney [34] proposed an alternative approach to assess the influence of small stress range cycles on fatigue damage. VA experimental results were analysed in terms of number of blocks to failure. With different test specimens, the minimum damaging stress range was found by gradually decreasing the lowest stress range of loading spectra and checking the effect on the number of blocks to failure: if the lowest stress range didn't damage then an increase of blocks to failure was expected. Concave spectra having constant relative fatigue damage, $(S_i^{m_1} \cdot n_i / S_{\max} \cdot 1.0)$, were used in this type of investigation. The results obtained for as-welded specimens tested with 4 different concave spectra, with a peak range at R=0, showed that stress range cycles down to 10MPa were fully damaging (suggesting the use of one-slope extrapolated S-N curve).

Zhang and Maddox [88] published results of an investigation of fatigue damage in welded joints under VA loading spectra. Two types of fillet welded connections were tested: 1) longitudinal non-load-carrying in-plane gusset; and 2) longitudinal non-load-carrying fillet welded joint. A concave stress spectrum, having relative stress ranges, $(S_i / S_{\max})$, that increase exponentially with decreasing cumulative number of cycles $N_{E,i}$ was used, as given below :

$$\ln(N_{E,i}) = 1 - \left(\frac{S_i}{S_{\max}} \right)^{\nu} \cdot \ln(N_{tot}), \text{ with } \nu < 1 \quad (\text{Eq. 30})$$

This stress spectrum type was used in order to make the results of VA tests more sensitive to small stress ranges. The VA test results were analysed using Miner's rule with three different S-N curves: 1) S-N curve with no-slope change; 2) S-N curve with slope change at the CAFL (at 10^7 cycles, according to BS standards); 3) S-N curve with cut-off at the CAFL. Comparison of number of block to failures and damage sums for spectra having different minimum stress ranges, allowed authors to conclude that stress ranges well below the CAFL were fully damaging.

Albrecht et al. [6] reported experimental CA and VA fatigue test results for 192 specimens from different types of welded joints; VA spectra were proportional to real truck traffic histograms which were recorded on several U.S. bridges. VA results were plotted in the

⁹ CAFL of the characteristic curve of studied detail, according to AASHTO specifications [4].

CA $\log(S-N)$ plane in terms of the RMC stress range of the spectrum $\left(S_{eq} = \left(\sum n_i / n_{tot} \cdot S_i^3 \right)^{\frac{1}{3}} \right)$. They found that: 1) VA and CA fatigue test data correlate well

in the FL region; 2) When the maximum stress range of the VA spectrum is equal to the CAFL, the equivalent VA stress range is equal to the VA fatigue limit (VAFL in AASHTO bridge design specifications); 3) When the percentage of VA stress range cycles below the CAFL increases, the CA S-N curve gradually bends and approaches the VAFL; and 4) Since the position of the VAFL depends on the type of loading spectrum it is not possible to establish a unique formulation for the VAFL; for loading spectra similar to the truck traffic proportional spectrum, it is safe to fix the VAFL at 50% of the CAFL.

In conclusion, studies presented above provide support to the use of one-slope S-N curve for fatigue analysis under VA loadings.

The second critical aspect in using Miner's rule is related to the choice of the critical value of the damage sum, D_c .

Miner and Palmgren [58] settled D_c to 1.0 by assuming that the fatigue damage corresponding to each stress cycle of a VA loading sequence is the same as that due to the same stress cycle under CA loading sequence. However, many studies have shown that the stress cycles of a VA loading sequences could be more damaging than the same stress cycles under CA loading, with the result that the Miner's rule can be unsafe in some cases.

In their investigation, Zhang and Maddox [88] considered three types of loading sequences: A) Constant maximum stress; B) Constant mean stress; and C) Constant minimum stress. In order to evaluate the accuracy of Miner's rule by eliminating influence of mean stress, CA S-N curves were assessed by mean of CA experimental tests at constant maximum stress (same condition as the sequence A). Results for both details showed a strong dependence of the damage sum at failure, $\sum n/N$ to the loading sequence: the Miner's rule was highly non conservative for all tests under sequence A ($\sum n/N \leq 0.4$), slightly non conservative for tests under sequence B ($\sum n/N \leq 0.8$) and conservative for tests under sequence C ($\sum n/N \geq 1.3$).

Results of fatigue VA tests on 3 types of welded connections (longitudinal non load carrying fillet welds [73],[28],[84],[13],[7],[12], welded cover plates [73], transverse non load carrying fillet welds [31],[12]), under random and block loading spectra, were re-analysed by Gurney [34]. The analysis gave following results: 1) For the longitudinal non-load carrying fillet weld, tested under Rayleigh and Laplace random loading spectra, $\sum n/N$ (computed using BS F mean curve) ranged from 0.14 to 2.89 with a mean of 0.63; 2) For the longitudinal non-load carrying fillet weld, tested under block loading spectra, $\sum n/N$ (computed using BS F mean curve) ranged from 0.71 to 3.65 with a mean of 1.04; 3) For the welded cover plate beams, tested under Rayleigh spectra, $\sum n/N$ (computed using BS G mean curve) ranged from 0.53 to 6.69 with a mean of 1.23; 4) For the transverse non-load carrying fillet weld, tested under block loading spectra, $\sum n/N$ (computed using BS F mean curve) ranged from 0.91 to 12.08 with a mean of 2.50. Results show that the damage sum at failure is highly variable and shall be treated as a random variable.

In his block program investigation, Gurney [34] observed a strong influence of the form of the loading spectrum on the value of damage sum at failure: wide band loading was more

damaging than narrow band loading¹⁰. This effect was more significant at $R=-1.0$ than at $R=0$. Gurney also observed that the block program investigation gave values of $\sum n/N$ considerably more optimistic than other fatigue test program involving other types of spectra. Gurney analysed also the influence of loading spectrum parameters on the fatigue strength of as-welded specimens which were tested at constant stress ratio, R . He found that: 1) For concave spectra (see Fig. 23 (a)), $\sum n/N$ did not depend on the shape parameter k ¹¹; 2) For convex (i.e. sometimes referred to as concave downwards) spectra (see Fig. 23 (b)) having block length smaller than 1000 cycles, $\sum n/N$ increased with k to a peak and then decreased; instead for convex spectra having block length bigger than 16250 cycles, $\sum n/N$ increased monotonically with k ; 3) For convex spectra, $\sum n/N$ increased with the block length; and 4) The clipping ratio¹² had little influence on $\sum n/N$.

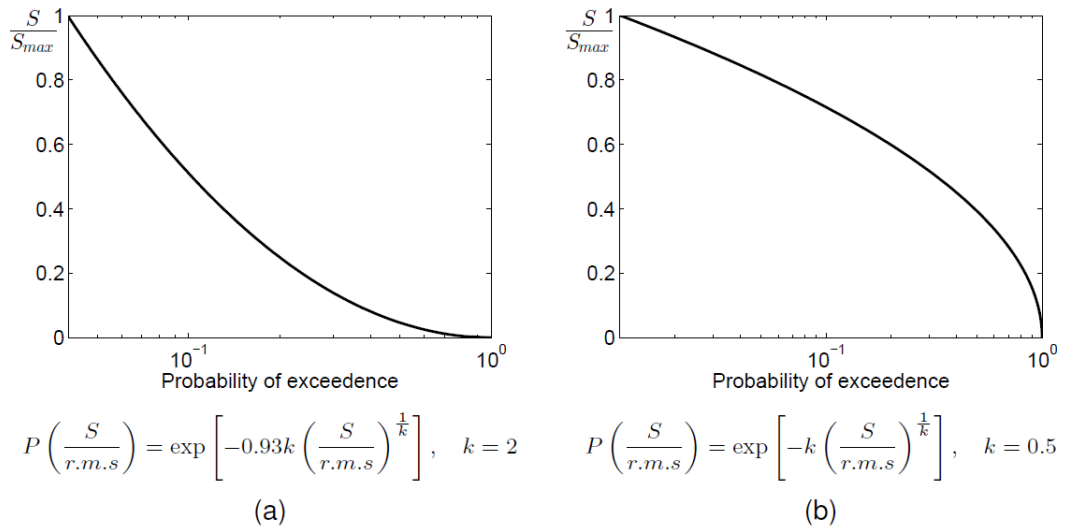


Fig. 23 Exceedance diagrams for Weibull spectrum with k shape factor. (a) Concave (concave upward) spectrum; (b) Convex (concave downward) spectrum.

Albrecht et al. [5] reviewed previous studies on VA amplitude fatigue in both finite life region and high cycle fatigue region. They proposed a simplified non-linear S-N model which approaches gradually the VAFL, for the fatigue assessment of steel bridges. They observed that the sequence of the blocks of the stress range spectrum, the spectrum size, the spectrum bandwidth and the minimum stress range did not have significant effect on $\sum n/N$.

¹⁰ According to [34] the most reliable method to define spectrum characteristics, in particular the bandwidth, is the Markov transition matrix, which has starting stress levels as row entries and finishing stress levels as column entries: for narrow band loadings all non zero entries are located close to the diagonal, while for wide band loadings non zero entries might be located anywhere. As often loading rates and ramp length are correlated, many authors consider that narrow band loadings correspond to narrow frequency loading histories.

¹¹ The shape parameter, k , characterizes the concavity of the loading spectrum (see Figure 5.1). The spectrum is concave for $k > 1$ and convex for $k < 1$.

¹² The clipping ratio is the ration between the maximum stress range and the r.m.s. of the process $\left(S_{\max} / \sqrt{\sum (S_i)^2 / n}\right)$.

The investigations listed above give an indication of influence of different VA loading spectrum parameters on the fatigue strength of analysed welded connections; however, these works only provide general trends and do not present a rigorous statistical approach which allows to make inference about the reduce slope, m_2 , and the critical damage sum, D_c . Moreover, they are affected by two substantial limitations: 1) The CA S-N curves used with Miner's rule have inaccurate definitions of CAFL and fatigue life scatter (especially in HCF region); 2) The approach used in these studies is qualitative and does not allow to formally characterize the randomness in the S-N curve variables and in the critical damage sum variable. In this Chapter a ML-MCS approach is proposed for estimation of: 1) Characteristic S-N curve under VA fatigue loadings; and 2) Critical value of damage sum at failure, D_c . The critical damage sum is modeled as a random variable whose parameters are included in the VA S-N model parameter vector. The slope of the characteristic S-N curve and the parameters of D_c distributions are estimated by using novel scheme which combines ML and MCS techniques by using the CA S-N stochastic model and the experimental data-set from VA fatigue tests. The novel contribution of this approach is summarized in the following points:

- The slope of the S-N curve below the CAFL, m_2 and the critical damage sum D_c are validated by using VA experimental results;
- The effect of the stress range cycles below the CAFL is not arbitrarily assumed but it's represented by the estimated m_2 and D_c ;
- Load sequence, stress ratio, spectrum shape, loading bandwidth and block length effects are represented in the variability of the D_c random variable.

The ML-MCS approach for estimation of VA S-N curves is implemented in `VAsamgener` and `VApstproc` modules of Matlab Toolbox TB1 (see Appendix III).

The Chapter is organized as follows:

- in Section 4.2.1 the ML-MCS approach for estimation of VA S-N curves is presented;
- in Section 4.3 the application of the ML-MCS approach on two study cases is presented;
- in Section 4.4 results of analysis of two considered study cases are presented;
- in Section 4.5 results are discussed.

4.2 New probabilistic approach

The VA S-N stochastic model is defined by the S-N curve under VA loadings (Equation 31) and by the fatigue failure condition (Equation 32):

$$Y = \begin{cases} m_0 + m_1 X + \epsilon(0, \exp(\sigma)), & \text{for } X > V \\ m_0 + V \Delta m + (m_1 - \Delta m) X + \epsilon(0, \exp(\sigma)), & \text{for } X \leq V \end{cases} \quad (\text{Eq. 31})$$

$$\sum_{i=1}^{n_{tot}} \frac{n_i}{\exp(Y(x_i))} = D_c(\mu_D, \sigma_D), \quad \text{if } \max(x_i) > V \quad (\text{Eq. 32})$$

where:

- m_0 and m_1 are the intercept and the slope of the S-N curve, which are defined in the CA S-N stochastic model;
- $\epsilon(0, \exp(\sigma))$ is the error term, which is defined in the CA S-N stochastic model;
- $V(\mu_V, \sigma_V)$ is the natural logarithm of the CAFL, which is defined in the CA S-N stochastic model;

- $\Delta m = m_1 - m_2$, where m_2 is the slope of the S-N curve below the CAFL;
- $D_c(\mu_D, \sigma_D)$ is the critical damage sum.

D_c is a random variable following a log-Normal (conventional choice) or a Weibull distribution with location parameter equal to μ_D and scale parameter equal to σ_D .

The VA S-N stochastic model is defined by the model parameter vector $\underline{\Theta} = (\underline{\theta}, \mu_D, \sigma_D)_{\Delta m}$ and by the covariance matrix $\underline{\Sigma}(\underline{\Theta})$.

4.2.1 ML-MCS estimation of VA S-N stochastic model

The VA S-N stochastic model is estimated with a novel ML-MCS scheme, using experimental VA fatigue results.

Let denote the VA fatigue data-set as follows:

$$(\varpi_t, y_t) \mid t=1 \dots n_{ts} \quad (\text{Eq. 33})$$

where ϖ_t is the t^{th} stress range spectrum, y_t is the natural logarithm of number cycles to the end of the t^{th} test and n_{ts} is the total number of tests.

The damage, d_t , associated with the t^{th} test, is:

$$d_t(\underline{\theta}, \Delta m) = \sum_{i=1}^{n_{tot1}} \frac{n_i}{\exp(Y(m_0, m_1, \sigma; x_i))} + \sum_{j=1}^{n_{tot2}} \frac{n_j}{\exp(Y(m_0, m_1, \sigma, V(\mu_V, \sigma_V), \Delta m; x_j))} \quad (\text{Eq. 34})$$

where n_{tot1} and n_{tot2} are respectively the number of stress range cycles above the CAFL and below the CAFL in the spectrum ϖ_t .

The fatigue failure condition is:

$$d_t(\underline{\theta}, \Delta m) = D_c(\mu_D, \sigma_D) \quad (\text{Eq. 35})$$

where the critical damage sum, D_c , is a random variable following a log-Normal or a Weibull distribution, with parameters μ_D and σ_D .

The following ML-MCS scheme is used to estimate the parameters of VA S-N stochastic model:

1. The experimental VA fatigue data-set is re-simulated η_{res} time. The re-simulated experimental data-set is:

$$(\varpi_t, y_t) \mid t=1, \dots, n_{ts} \cdot \eta_{res}$$

2. One vector $\tilde{\underline{\theta}}_k$ is sampled from the CA S-N stochastic model $[\underline{\theta}, \underline{\rho}]$; then, one S-N curve is sampled by using the sampled $\tilde{\underline{\theta}}_k$. The epistemic uncertainty of S-N model parameters is taken into account in the $\tilde{\underline{\theta}}_k$ sampling, while the aleatory uncertainty of

the CAFL and the aleatory uncertainty of the fatigue life are taken into account in the S-N curve sampling.

For a given value of parameter $\Delta\tilde{m}$ the negative log-Likelihood of the re-simulated sample is computed as follows:

$$nLL_k(\mu_{D_k}, \sigma_{D_k})|_{\Delta\tilde{m}} = - \sum_{j=1}^{\eta_{res}} \left[\sum_{t=1}^{n_f} \ln(f_D(\mu_{D_k}, \sigma_{D_k}, \Delta\tilde{m}; d_t)) + \sum_{t=1}^{n_r} \ln(1 - F_D(\mu_{D_k}, \sigma_{D_k}, \Delta\tilde{m}; d_t)) \right]$$

where n_f is the number of failures, n_r is the number of run-outs, f_D is the pdf of the random variable D_c and F_D is the cdf of the random variable D_c .

3. ML estimates of parameters μ_D and σ_D are computed by minimizing the negative log-Likelihood of the re-simulated sample:

$$(\hat{\mu}_D, \hat{\sigma}_D)_k = \arg \min_{\mu_{D_k}, \sigma_{D_k}} (nLL_k(\mu_{D_k}, \sigma_{D_k})|_{\Delta\tilde{m}})$$

4. Steps 1-3 are iterated η_{iter} times.

The $\underline{G}$ matrix and the $\underline{L}$ vector are built:

$$\begin{aligned} \underline{G} &= [(\theta)_k \quad (\hat{\mu}_D)_k \quad (\hat{\sigma}_D)_k]_{\Delta\tilde{m}} \quad \text{for } k = 1, \dots, \eta_{iter} \\ \underline{L} &= \left[\left(n\hat{LL} \right)_k \right]_{\Delta\tilde{m}} \quad \text{for } k = 1, \dots, \eta_{iter} \end{aligned}$$

5. Under the assumption of asymptotic normality of ML estimators¹³ $\hat{\mu}_D$; $\hat{\sigma}_D$; $n\hat{LL}$, it is observed that: 1) The matrix $\underline{G}$ contains η_{iter} realizations of the multivariate Normal random variable $(\theta; \hat{\mu}_D; \hat{\sigma}_D)$; and 2) The vector $\underline{L}$ contains η_{iter} realizations of the Normal random variable $n\hat{LL}$.

For a given value of parameter $\Delta\tilde{m}$, the VA S-N model parameter vector, $\underline{\Theta}$, and the covariance matrix, $\underline{\Sigma}(\underline{\Theta})$ are computed as follows:

$$\begin{aligned} \underline{\Theta}(\Delta\tilde{m}) &= E[\underline{G}]_{\Delta\tilde{m}} \\ \underline{\Sigma}(\Delta\tilde{m}) &= E \left[(\underline{G} - E(\underline{G})) \cdot (\underline{G} - E(\underline{G}))^T \right]_{\Delta\tilde{m}} \end{aligned}$$

The mean of the $n\hat{LL}$ distribution is indicated as $\overline{n\hat{LL}}$.

6. Steps 1-5 are repeated for different values of $\Delta\tilde{m}$. The ML estimate of the parameter Δm is chosen as the value which minimizes the mean of the $n\hat{LL}$ distribution¹⁴:

$$\Delta\hat{m} = \operatorname{argmin}_{\Delta\tilde{m}} \left(\overline{n\hat{LL}}(\Delta\tilde{m}) \right)$$

¹³ This hypothesis is verified by using probability plots.

¹⁴ The choice of computing different $\underline{\Theta}(\Delta\tilde{m})$ vectors by assuming different values of $\Delta\tilde{m}$ (steps 1-5) and then to proceed in searching for the value $\Delta\hat{m}$ which minimizes $\overline{n\hat{LL}}(\Delta\hat{m})$ (step 6) was made because the initial choice of including Δm in the model parameter vector did not work in terms of numerical optimization.

The ML estimate of the VA S-N model parameter vector and the covariance matrix are:

$$\begin{aligned}\hat{\underline{\Theta}} &= \underline{\Theta}(\Delta\hat{m}) \\ \hat{\underline{\Sigma}} &= \underline{\Sigma}(\Delta\hat{m})\end{aligned}$$

A sensitivity study is performed in order to determine the parameters η_{iter} and η_{res} that have to be used in the ML-MCS scheme described above. Since the $\overline{nLL}$ is not a deterministic parameter, the minimum search algorithm includes a Box-plot test to ensure that $\overline{nLL}(\Delta\hat{m})$ is lower than $\overline{nLL}(\Delta\hat{m} \pm 1)$ at 5% significance level. The ML-MCS based characteristic SN curve is defined as follows:

$$y_{.05} = \begin{cases} a_{.05} + \hat{m}_1 x, & \text{for } x > v_{.05} \\ a_{.05} + v_{.05} \Delta\hat{m} + (\hat{m}_1 - \Delta\hat{m})x, & \text{for } x \leq v_{.05} \end{cases} \quad (\text{Eq. 36})$$

The characteristic S-N curve in Equation 36 has to be used in combination with following damage accumulation rule:

$$\sum_{i=1}^{n_{tot}} \frac{n_i}{\exp(y_{.05}(x_i))} \leq d_{c,.5}, \quad \text{with } d_{c,.5} = F_D^{-1}(0.5, \mu_D, \sigma_D) \quad (\text{Eq. 37})$$

where F_D^{-1} is the inverse of the cdf of the critical damage, D_c .

The ML-MCS-based VA characteristic S-N curve has to be re-scaled for direct comparison with VA characteristic S-N curves from the standards, which consider $d_{c,.5} = 1.0$.

The ML-MCS-based re-scaled VA characteristic S-N curve (see Fig. 24) is defined as follows:

$$y_{.05}^r = \begin{cases} a_{.05} - \ln(1/d_{c,.5}) + \hat{m}_1 x, & \text{for } x > v_{.05} \\ a_{.05} - \ln(1/d_{c,.5}) + v_{.05} \Delta\hat{m} + (\hat{m}_1 - \Delta\hat{m})x, & \text{for } x \leq v_{.05} \end{cases} \quad (\text{Eq. 38})$$

Three cases are considered for the cut-off stress range, S_{co} : 1) No cut-off stress range; 2) Cut-off stress range at 14.6 MPa, which corresponds to the cut-off stress range of the lowest FAT category in EN 1993-1-9; 3) Cut-off stress range at 50% of the CAFL, according to [83].

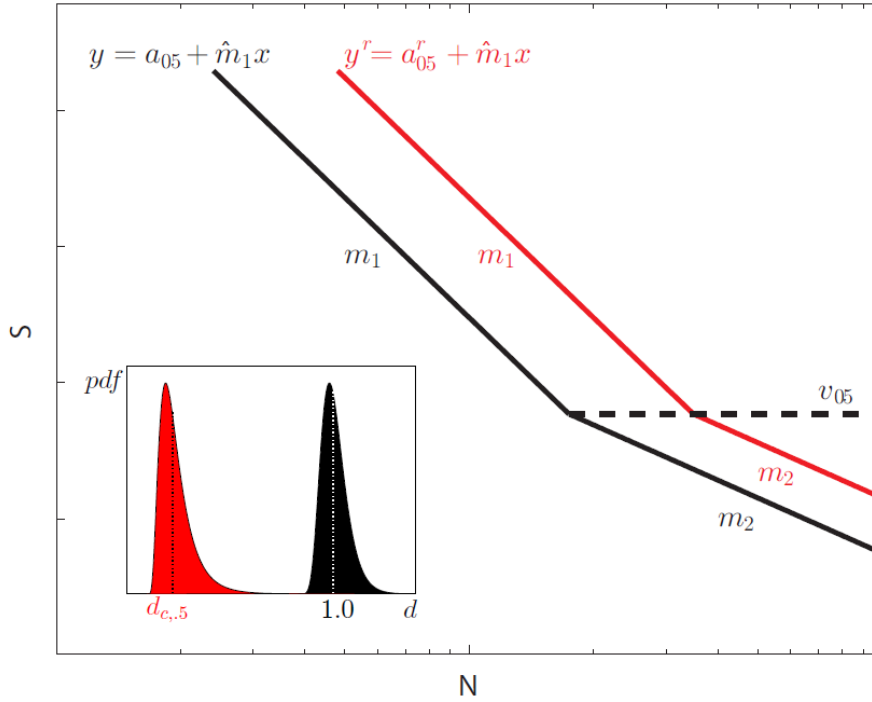


Fig. 24 ML-MCS-based VA characteristic S-N curve and re-scaled VA characteristic S-N curve.

4.3 Study cases

The ML-MCS approach was applied on the two fatigue data-sets analysed in Chapter 3.

4.3.1 Cover Plate

VA experimental data-set for cover plate detail is made of 32 data points (see Tab. 6) from testing of end-welded cover plate beam, under 5 different wide-band Rayleigh-type stress range spectra with a block length ranging between 1001 and 10001 cycles [28]. The thickness of the beam flange and of the cover plate is 25.4mm: the detail is classified as FAT 45 according to EN1993-1-9 [74], as FAT 50 according to IIW recommendations [42], and as FAT40 (with CAFL=18MPa) according to AASHTO bridge design specifications [4]. The CA S-N stochastic model estimated by using the cover plate CA data-set with size effect correction is used for ML-MCS estimation of VA S-N stochastic model.

4.3.2 In-plane gusset

VA experimental data-set for in-plane gusset detail is made of 21 data points (see Tab. 7) from testing of welded in-plane gussets, under 9 different three-blocks stress range spectra with a block length ranging between 800 and $152 \cdot 10^3$ cycles and constant relative damage $(S_i^{m_1} \cdot n_i / S_{\max})$ (ICOM tests, [10]). The attachment length, L , is equal to 150mm: the detail is classified as FAT40 according to EN1993-1-9 [74], as FAT50 according to IIW recommendations [42], and as FAT 56 (with CAFL=31MPa) according to AASHTO bridge design specifications [4].

Tab. 6 Cover plate, VA experimental data-set

S_i [MPa]	n_i				
8				170	
9	73			1200	
10			170	3600	
11	190		1200	1600	
12	237		1700	1300	
13			3500	800	
14	207	73	1300	900	
15			800	200	
16	146		600	100	
17		190	300	60	
18	83		200	60	
19	40	237	100	10	
20			60		
21	16		40		
22		207	20		
23	5		10		
25	2	146			
26				1	670
27					600
28		83			
29	1				300
30		40			200
31			1		
32					100
33		16			
34					60
35					40
36		5			
37					20
38		2			10
51					1
58		1			
$n_{fail} \cdot 10^6 \rightarrow$	> 107.2	120	> 104	> 109	34.7
	> 107.2	120	> 104	> 109	> 34.7
	> 107.2	> 120	> 104	> 109	> 34.7
	> 107.2	> 120	104	> 109	> 34.7
	> 107.2	120			34.7
	> 107.2	120			> 34.7
	> 107.2	> 120			32.0
	> 107.2	> 120			> 34.7
	> 107.2	> 120			> 34.7

The column S_i represents the stress ranges of the loading spectrum. The columns n_i represent the number of cycles associated to the stress ranges of the first column; each column of n_i represents a different spectrum. In the lower part of the table, the number of cycles to failure, n_{fail} , are presented; the values which are presented in the same column were obtained under the same loading spectrum. The symbol $>$ is used to indicate a run-out.

Tab. 7 In-plane gusset, VA experimental data-set

S_i [MPa]	n_i								
30				3700	29600	15200	6400		$152 \cdot 10^3$
40			3700	1560	12500	6400	2700	$64 \cdot 10^3$	$64 \cdot 10^3$
50		3278	1383	800				$33 \cdot 10^3$	
60	3701	1896	800				800		
70									
80	1562	800				800		$8 \cdot 10^3$	$8 \cdot 10^3$
90									
100	800				800				
$n_{fail} \cdot 10^6 \rightarrow$	1.11	1.61	4.61	> 40	6.78	13.2	9.52	3.17	15.08
	1.18	2.43	4.73	10.4	9.64	9.15	> 22.1	3.65	
				14.0			16.1	3.67	
							> 29.6		

The column S_i represents the stress ranges of the loading spectrum. The columns n_i represent the number of cycles associated to the stress ranges of the first column; each column of n_i represents a different spectrum. In the lower part of the table, the number of cycles to failure, n_{fail} , are presented; the values which are presented in the same column were obtained under the same loading spectrum. The symbol $>$ is used to indicate a run-out.

4.4 Results

4.4.1 Sensitivity study

VA S-N models were estimated for the two analysed VA data-sets, using the ML-MCS scheme presented in Section 4.2.1. Three cut-off cases (No cut-off, $S_{co} = 14.6$ MPa, $S_{co} = 0.5 \cdot \text{CAFL}$) and two critical damage sum distributions ($D_c = \log \mathcal{N}$, $D_c = \mathcal{W}$) were considered.

In order to choose the sampling parameters η_{iter} , η_{res} of ML-MCS scheme, a sensitivity study was carried out, using cover plate VA data-set with combination *No cut-off* / $D_c = \mathcal{W}$. For $\eta_{iter} = [100; 1000; 2000; 5000]$, $\eta_{res} = [10; 50; 100; 200]$, and $\Delta m = \Delta \hat{m} = 8$, the parameter $\overline{n\hat{LL}}$ was sampled 16 times and $\Delta n\hat{LL} = \left(\overline{n\hat{LL}_{\max}} / \overline{n\hat{LL}_{\min}} - 1 \right)$ was computed. Fig. 25 shows that for $\eta_{iter} = 5000$ and $\eta_{res} = 100$, then $\Delta n\hat{LL}$ is lower than 0.2%: for a negative log-Likelihood equal to 1000,

the variation in $n\hat{LL}$ is close to unity, which ensures an accurate ML estimation of the Δm parameter for a Δm step equal to 1. The combination $\eta_{iter} = 5000$; $\eta_{res} = 100$ was chosen for the estimation of VA S-N models.

The ML estimates of parameters μ_D ; σ_D , Δm and the value of the negative log-Likelihood at ML estimates, for the cover-plate data-set, are shown in Tab. 8. The combination *no cut-off* / $D_c = \mathcal{W}$ gives the best fit in terms of negative log-Likelihood. Between the two cut-off cases, the case $S_{co} = 14.6\text{MPa}$ gives the best fit in terms of negative log-Likelihood. Fig. 26 shows that for the cover plate data-set there is a strong negative linear correlation between the parameters of the critical damage sum, D_c , and that there is no linear correlation between the parameters of the S-N curve and the parameters of the critical damage sum, D_c .

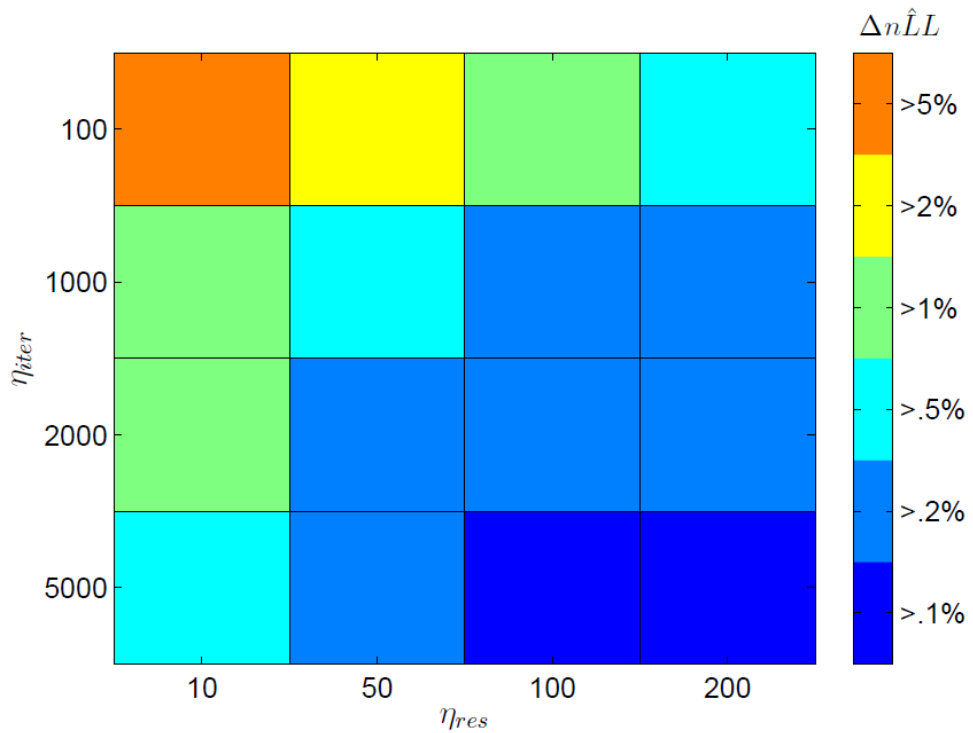


Fig. 25 Sensitivity study on ML-MCS scheme parameters, Cover plate, no cut-off / $D_c = \mathcal{W}$.

Tab. 8 Cover plate, VA S-N curve parameters

Cut-off case	I	D_c	$\hat{m}_0$	$\hat{m}_1$	$\hat{\sigma}$	$\hat{\mu}_V$	$\hat{\sigma}_V$	$\hat{\mu}_D$	$\hat{\sigma}_D$	$\Delta\hat{m}$	$\overline{n\hat{L}L}$
no cut-off		$\log \mathcal{N}$	28.65	-3.42	-0.54	3.45	-1.62	0.50	1.46	7	1431
		$\mathcal{W}$						1.70	1.40	8	1183
14.6 MPa		$\log \mathcal{N}$						0.50	0.87	4	1339
		$\mathcal{W}$						1.75	1.61	7	1190
0.5 · CAFL		$\log \mathcal{N}$						0.59	1.24	6	1482
		$\mathcal{W}$						2.13	1.28	8	1324

$$\underline{\underline{c}} = \begin{bmatrix} 1.00 & -1.00 & 0.08 & -0.05 & 0.06 & -0.03 & 0.01 \\ \vdots & 1.00 & -0.07 & 0.05 & -0.06 & 0.02 & 0.01 \\ \vdots & \vdots & 1.00 & -0.04 & 0.04 & 0.02 & 0.00 \\ \vdots & \vdots & \vdots & 1.00 & -0.03 & -0.00 & -0.01 \\ \vdots & \vdots & \vdots & \vdots & 1.00 & 0.03 & -0.00 \\ \vdots & \vdots & \vdots & \vdots & \vdots & 1.00 & -0.71 \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & 1.00 \end{bmatrix}$$

Fig. 26 Cover plate, VA S-N model correlation matrix, no cut-off / $D_c = \mathcal{W}$.**Tab. 9** In-plane gusset, VA S-N curve parameters

Cut-off case	I	D_c	$\hat{m}_0$	$\hat{m}_1$	$\hat{\sigma}$	$\hat{\mu}_V$	$\hat{\sigma}_V$	$\hat{\mu}_D$	$\hat{\sigma}_D$	$\Delta\hat{m}$	$\overline{n\hat{L}L}$
no cut-off		$\log \mathcal{N}$	25.77	-2.67	-1.05	3.86	-1.67	-0.31	0.60	5	1009
		$\mathcal{W}$						1.00	1.48	5	1343
14.6 MPa		$\log \mathcal{N}$						-0.30	0.60	5	1009
		$\mathcal{W}$						1.00	1.48	5	1343
0.5 · CAFL		$\log \mathcal{N}$						-0.26	0.61	5	1117
		$\mathcal{W}$						1.07	1.40	5	1486

$$\underline{\underline{c}} = \begin{bmatrix} 1.00 & -1.00 & 0.00 & -0.05 & 0.06 & -0.01 & 0.01 \\ \vdots & 1.00 & -0.00 & 0.05 & -0.06 & 0.00 & 0.00 \\ \vdots & \vdots & 1.00 & -0.01 & 0.01 & 0.01 & 0.02 \\ \vdots & \vdots & \vdots & 1.00 & -0.62 & -0.08 & -0.02 \\ \vdots & \vdots & \vdots & \vdots & 1.00 & 0.04 & 0.02 \\ \vdots & \vdots & \vdots & \vdots & \vdots & 1.00 & 0.37 \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots & 1.00 \end{bmatrix}$$

Fig. 27 In-plane gusset, VA S-N model correlation matrix, no cut-off / $D_c = \mathcal{W}$.

The ML estimates of parameters μ_D ; σ_D , Δm and the value of the negative log-Likelihood at ML estimates, for the in-plane gusset data-set, are shown in Tab. 9. The combination (no cut-off/ $D_c = \log \mathcal{N}$) gives the best fit in terms of negative log-Likelihood. Between the two cut-off cases, the case $S_{co} = 14.6$ MPa gives the best fit in terms of negative log-Likelihood. Fig. 27 shows that for the in-plane gusset data-set there is a weak negative linear correlation between the parameters of the critical damage sum, D_c , and there is no linear correlation between the parameters of the S-N curve and the parameters of the critical damage sum, D_c .

The assumption of asymptotic normality of $\hat{\mu}_D$, $\hat{\sigma}_D$, $n\hat{LL}$ estimators (see step 5 of the ML-MCS approach in Section 4.2.1) is verified by: 1) Comparing the η_{iter} sampled value of $\hat{\mu}_D$, $\hat{\sigma}_D$, $n\hat{LL}$ with theoretical normal pdf; and 2) Producing probability plots for the η_{iter} sampled value of $\hat{\mu}_D$, $\hat{\sigma}_D$, $n\hat{LL}$. Density plots in Fig. 28, Fig. 30, Fig. 32, Fig. 34, Fig. 36, Fig. 38 and probability plots in Fig. 29, Fig. 31, Fig. 33, Fig. 35, Fig. 37, Fig. 39 support the hypothesis of asymptotic normality of the multivariate vector $(\hat{\mu}_D, \hat{\sigma}_D, n\hat{LL})$ for both the cover-plate data-set and the in-plane gusset data-set.

For the cover plate data-set and no cut-off case, the graph $\overline{n\hat{LL}} - \Delta m$ shows that $D_c = \mathcal{W}$ gives the best fit in terms of Likelihood (see Fig. 40). The ML estimate of the slope range, $\Delta\hat{m}$, was estimated by producing notched box plots at the four Δm values which gave the four minimum values of $\overline{n\hat{LL}}$. Fig. 41 shows that box-plot notches at $\Delta m = 8$ and $\Delta m = 9$ overlap, indicating that $\overline{n\hat{LL}}(\Delta m = 9)$ is not significantly lower than $\overline{n\hat{LL}}(\Delta m = 8)$ at the 5% significance level: $\Delta\hat{m} = 8$ is conservatively taken as ML estimate of the slope range.

For the in-plane gusset data-set and no cut-off case, the graph $\overline{n\hat{LL}} - \Delta m$ shows that $D_c = \log \mathcal{N}$ gives the best fit in terms of Likelihood (see Fig. 42). Fig. 43 shows that box-plot notches at $\Delta m = 4$ and $\Delta m = 5$ do not overlap, indicating that $\overline{n\hat{LL}}(\Delta m = 5)$ is significantly lower than $\overline{n\hat{LL}}(\Delta m = 4)$ at the 5% significance level: $\Delta\hat{m} = 5$ is taken as ML estimate of the slope range.

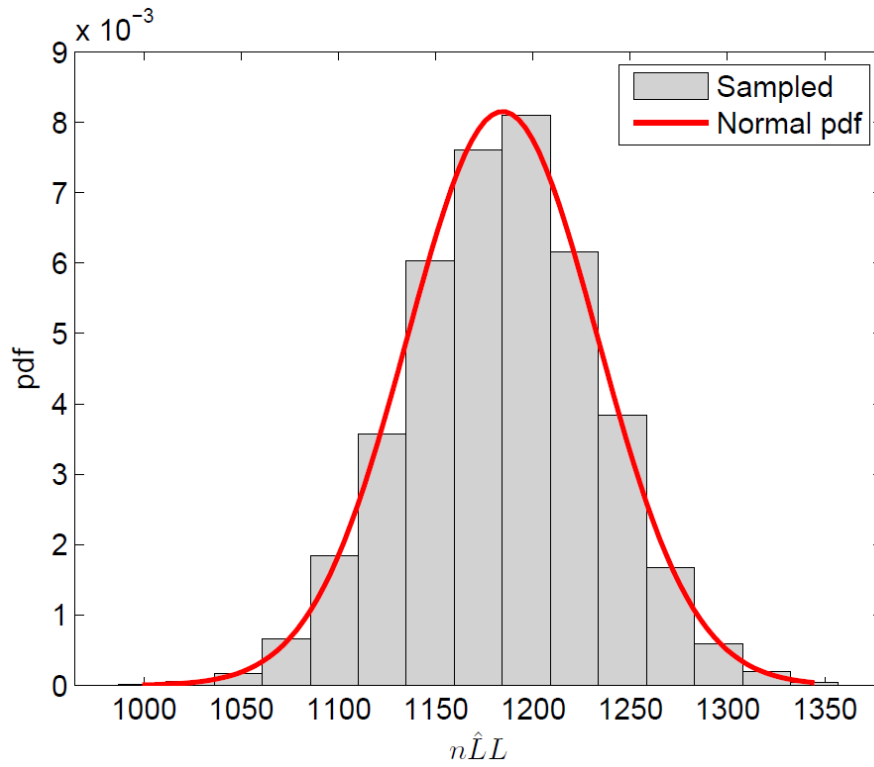


Fig. 28 Cover plate, pdf of VA neg. log-Likelihood, no cut-off / $D_c = \mathcal{W}$.

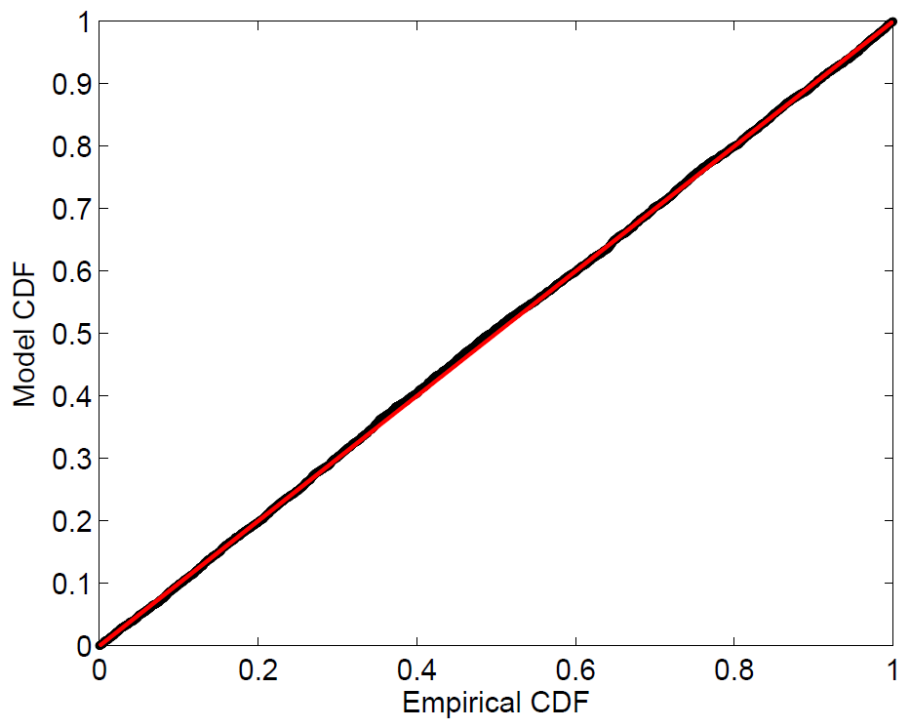


Fig. 29 Cover plate, Probability plot of VA neg. log-Likelihood, no cut-off / $D_c = \mathcal{W}$.

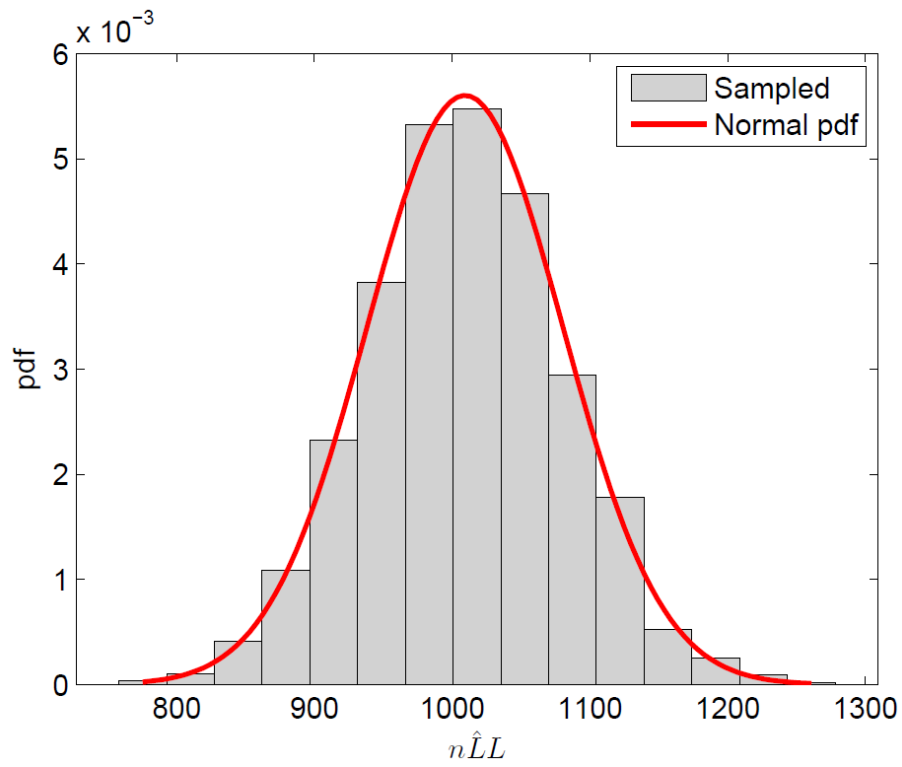


Fig. 30 In-plane gusset, pdf of VA neg. log-Likelihood, no cut-off / $D_c = \log \mathcal{N}$.

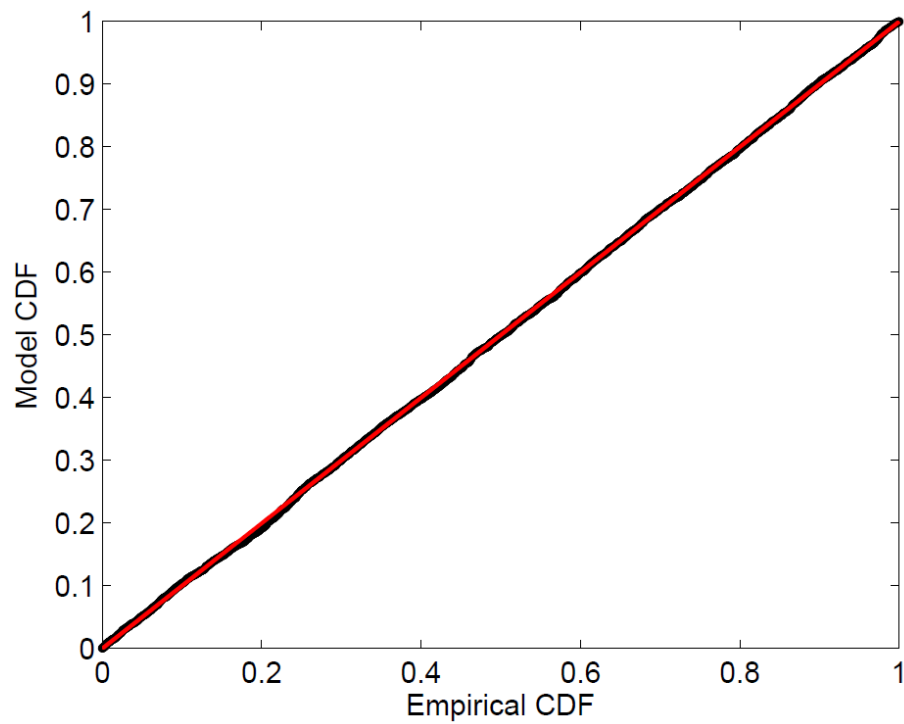


Fig. 31 In-plane gusset, Probability plot of VA neg. log-Likelihood, no cut-off / $D_c = \log \mathcal{N}$.

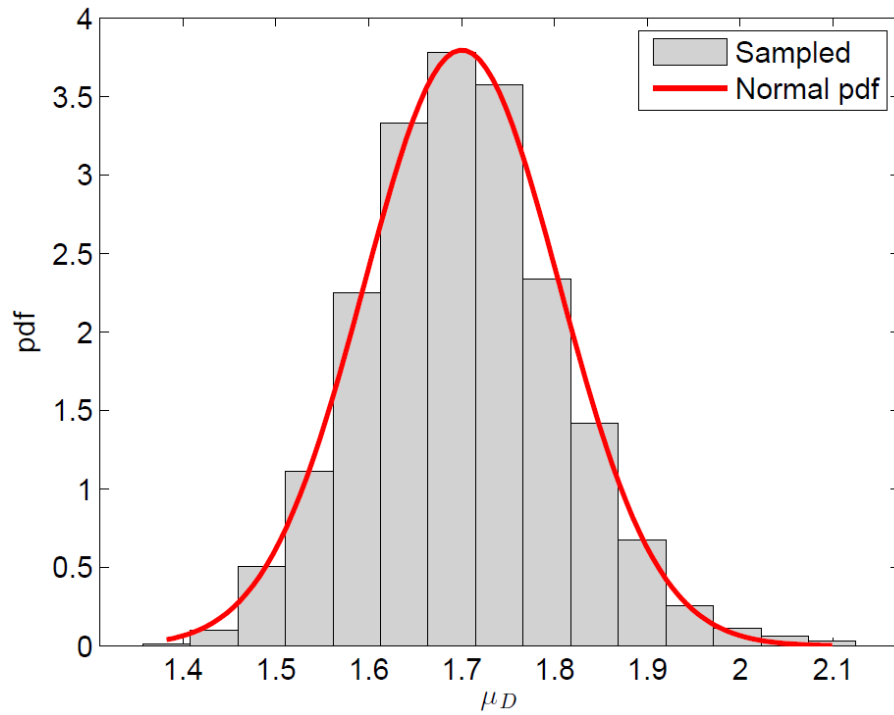


Fig. 32 Cover plate, pdf of the parameter μ_D , no cut-off / $D_c = \mathcal{W}$.

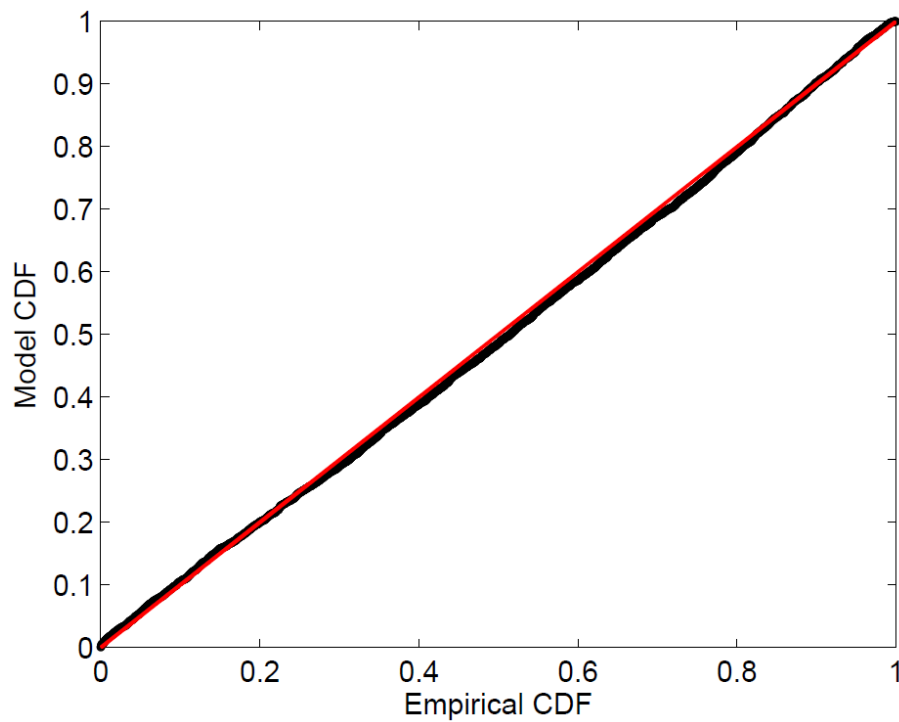


Fig. 33 Cover plate, Probability plot of the parameter μ_D , no cut-off / $D_c = \mathcal{W}$.

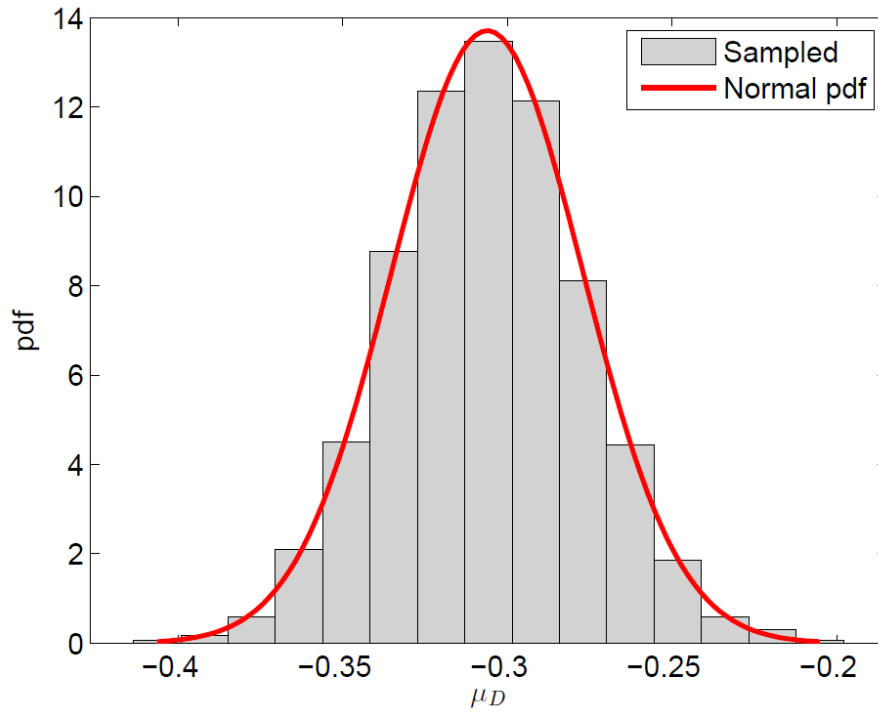


Fig. 34 In-plane gusset, pdf of the parameter μ_D , no cut-off / $D_c = \log \mathcal{N}$.

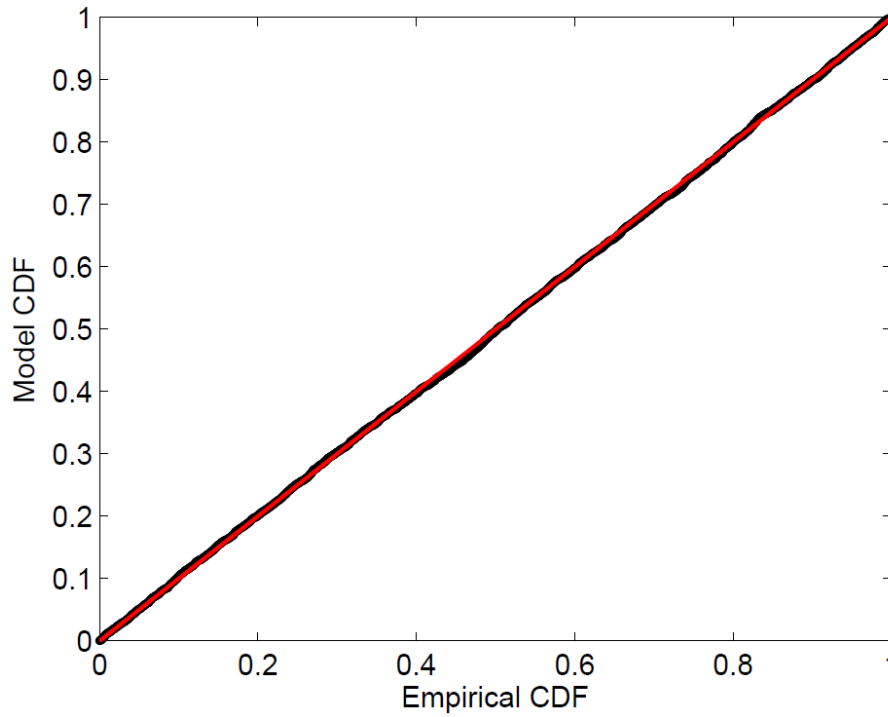


Fig. 35 In-plane gusset, Probability plot of the parameter μ_D , no cut-off / $D_c = \log \mathcal{N}$.

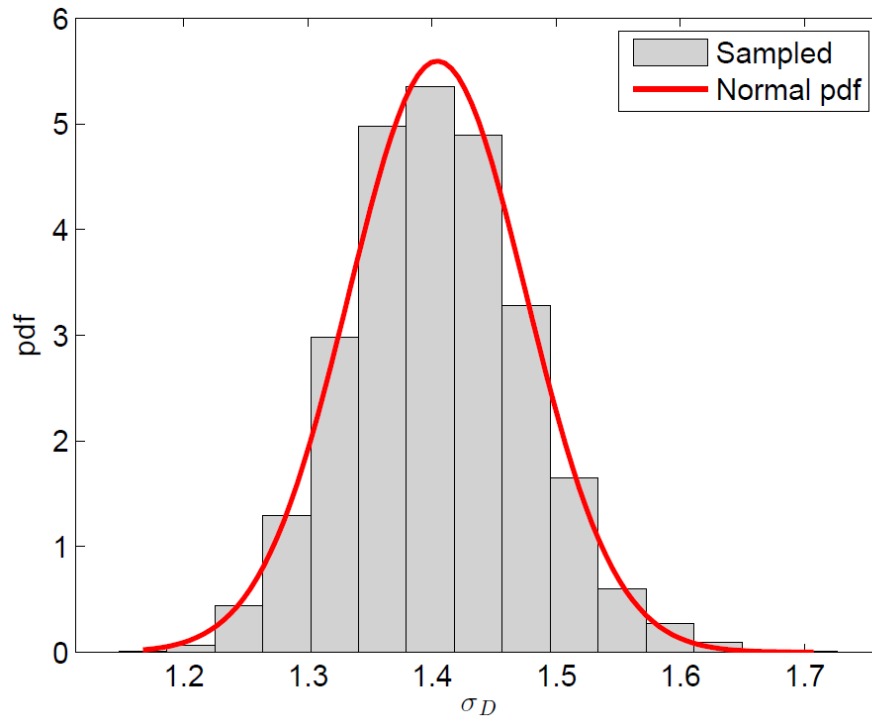


Fig. 36 Cover plate, pdf of the parameter σ_D , no cut-off / $D_c = \mathcal{W}$.

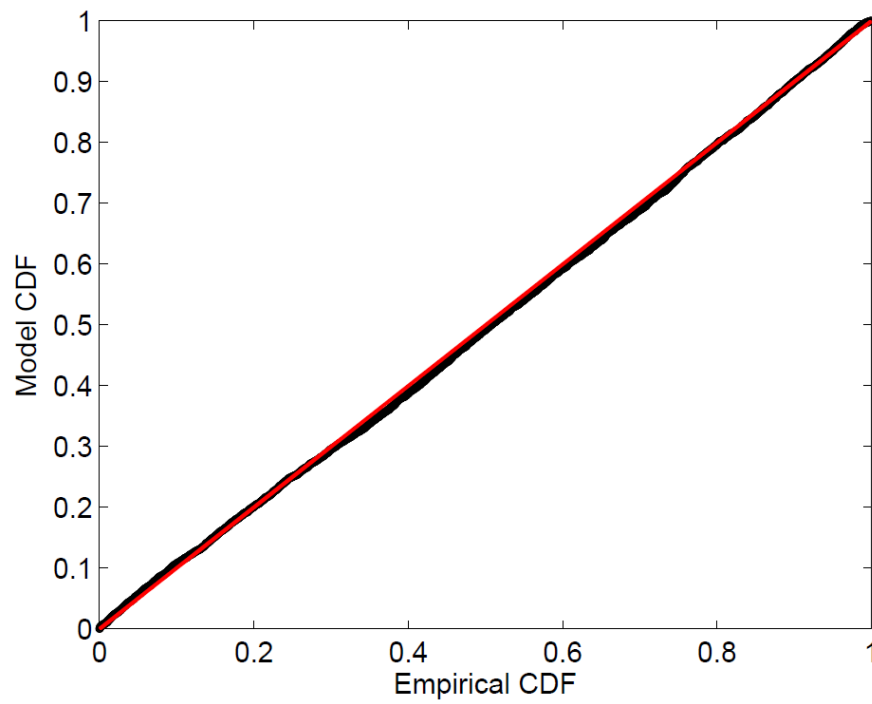


Fig. 37 Cover plate, Probability plot of the parameter σ_D , no cut-off / $D_c = \mathcal{W}$.

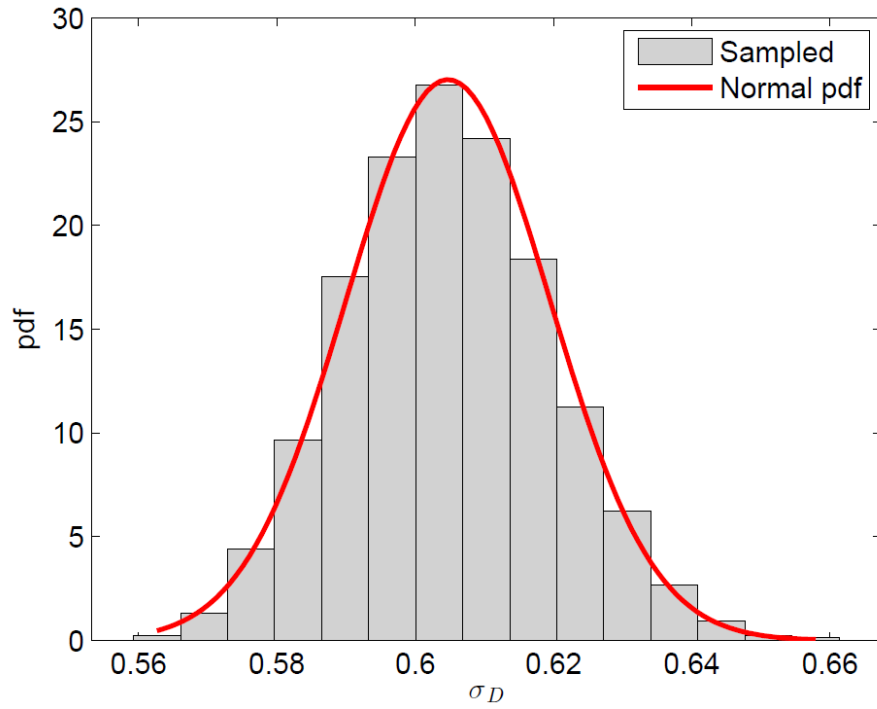


Fig. 38 In-plane gusset, pdf of the parameter σ_D , no cut-off / $D_c = \log \mathcal{N}$.

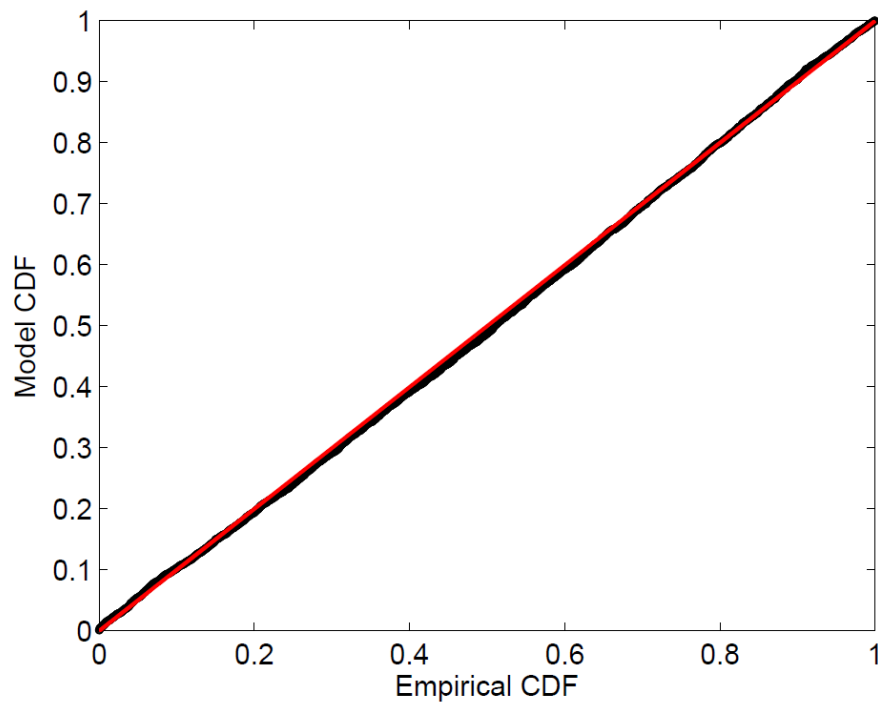


Fig. 39 In-plane gusset, Probability plot of the parameter σ_D , no cut-off / $D_c = \log \mathcal{N}$.

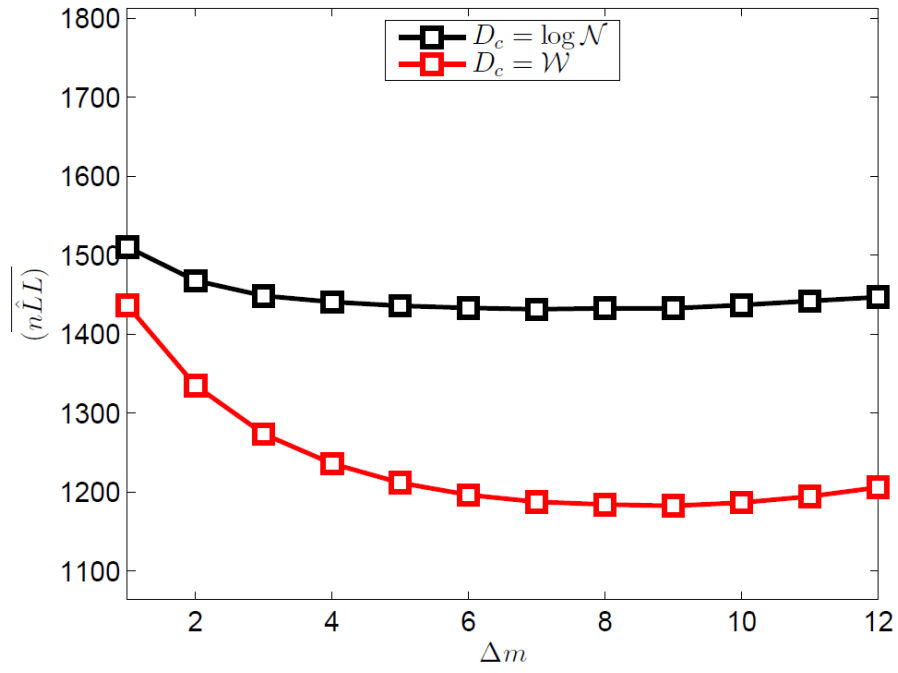


Fig. 40 Cover plate, negative log-Likelihood plot, no cut-off.

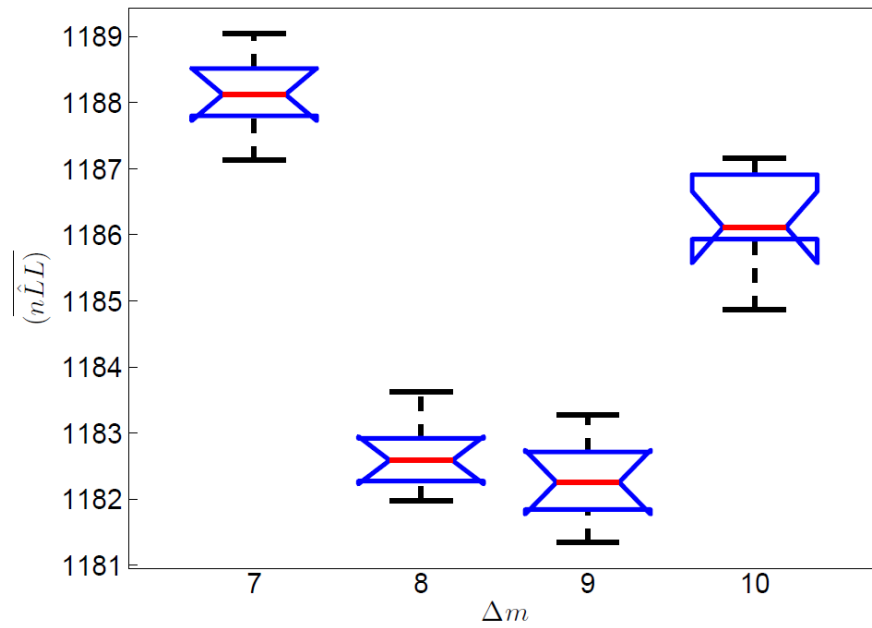


Fig. 41 Cover plate, negative log-Likelihood notched box plot, no cut-off / $D_c = \mathcal{W}$.

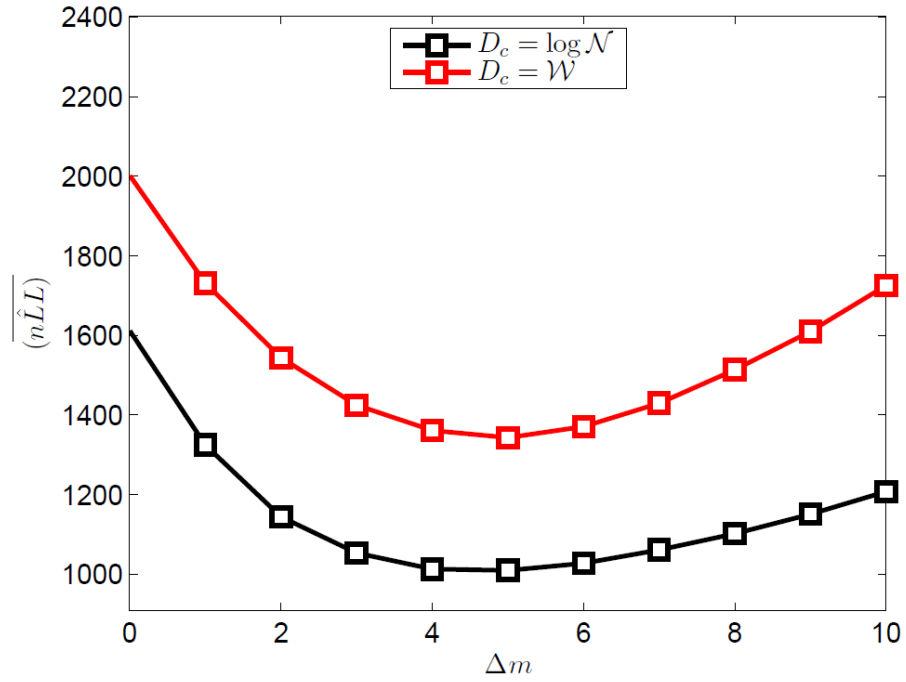


Fig. 42 In-plane gusset, negative log-Likelihood plot, no cut-off.

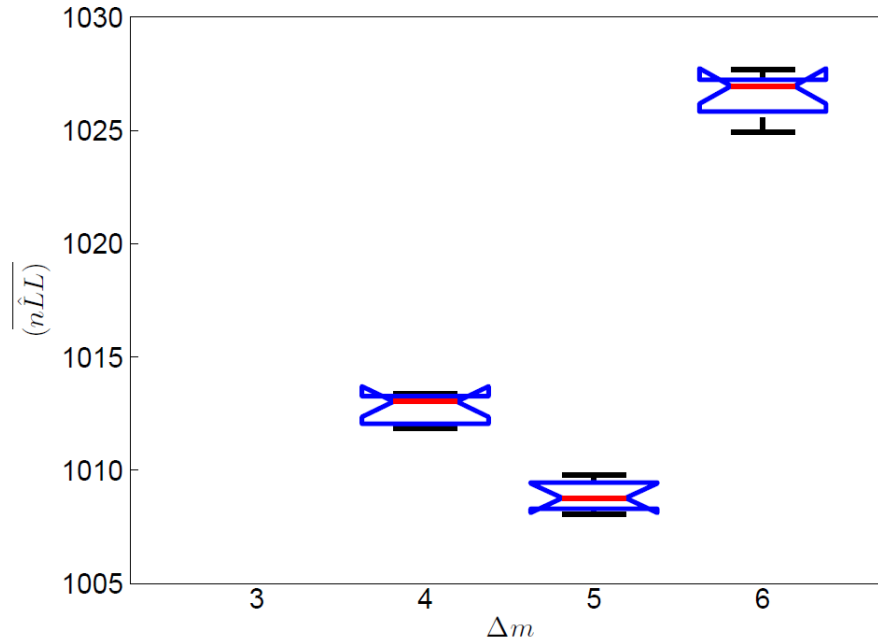
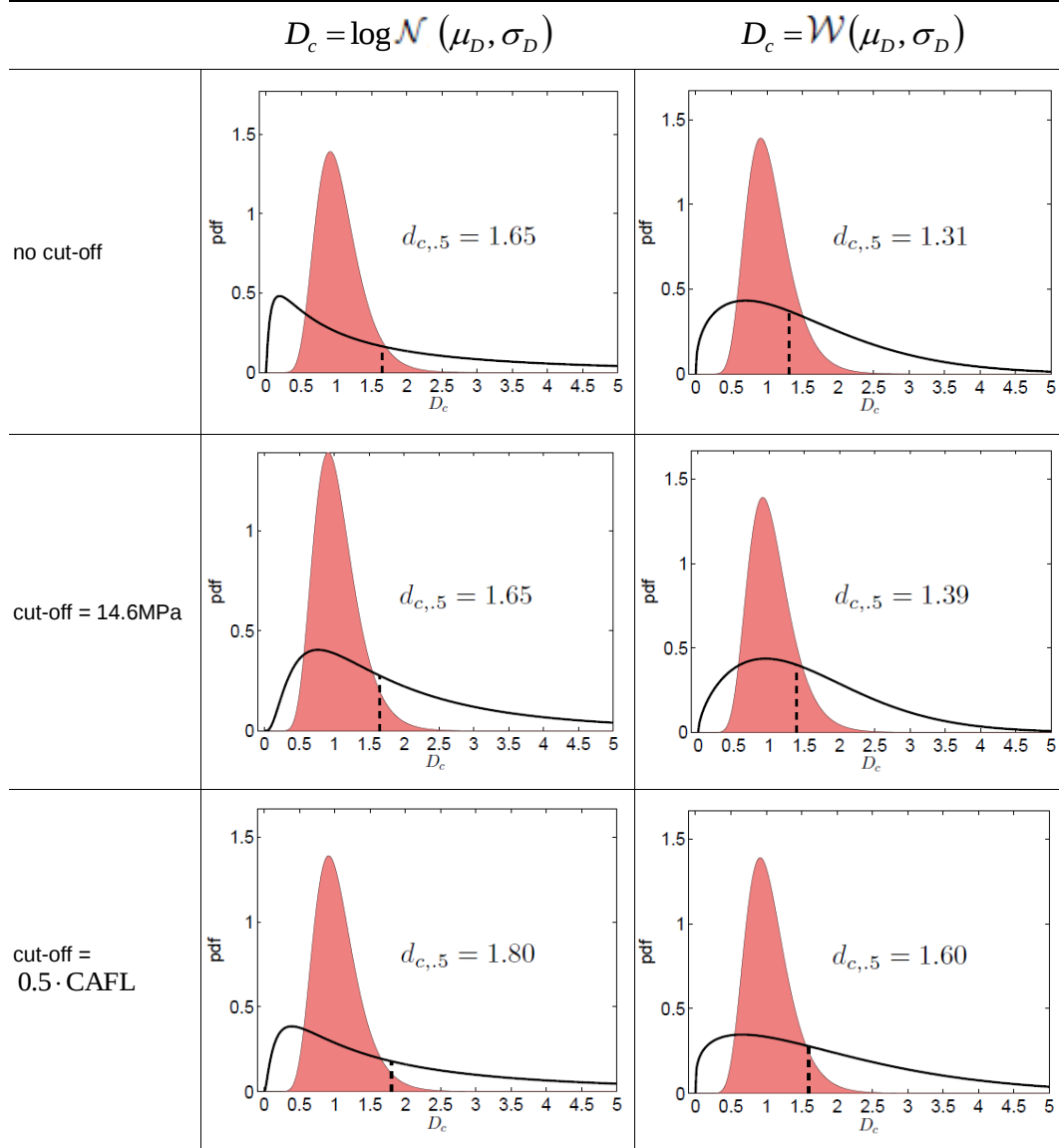


Fig. 43 In-plane gusset, negative log-Likelihood notched box plot, no cut-off / $D_c = \log \mathcal{N}$.

Tab. 10 and Tab. 11 show the D_c pdfs, corresponding to the six cut-off / D_c distribution combinations, for the cover-plate data-set and the in-plane gusset data-set respectively. The ML-MCS-based D_c distribution is compared to the conventional $\log \mathcal{N}(0, 0.3)$ damage distribution (red shaded pdf, gray on black/white copy); for each combination, the median value of the ML-MCS-based D_c distribution is also shown.

Tab. 10 Cover plate, D_c distributions. Shaded red pdf is the pdf of $\log \mathcal{N}(0, 0.3)$ distrib.



Tab. 11 In-plane gusset, D_c distributions. Shaded red pdf is the pdf of $\log \mathcal{N}(0, 0.3)$ distrib.

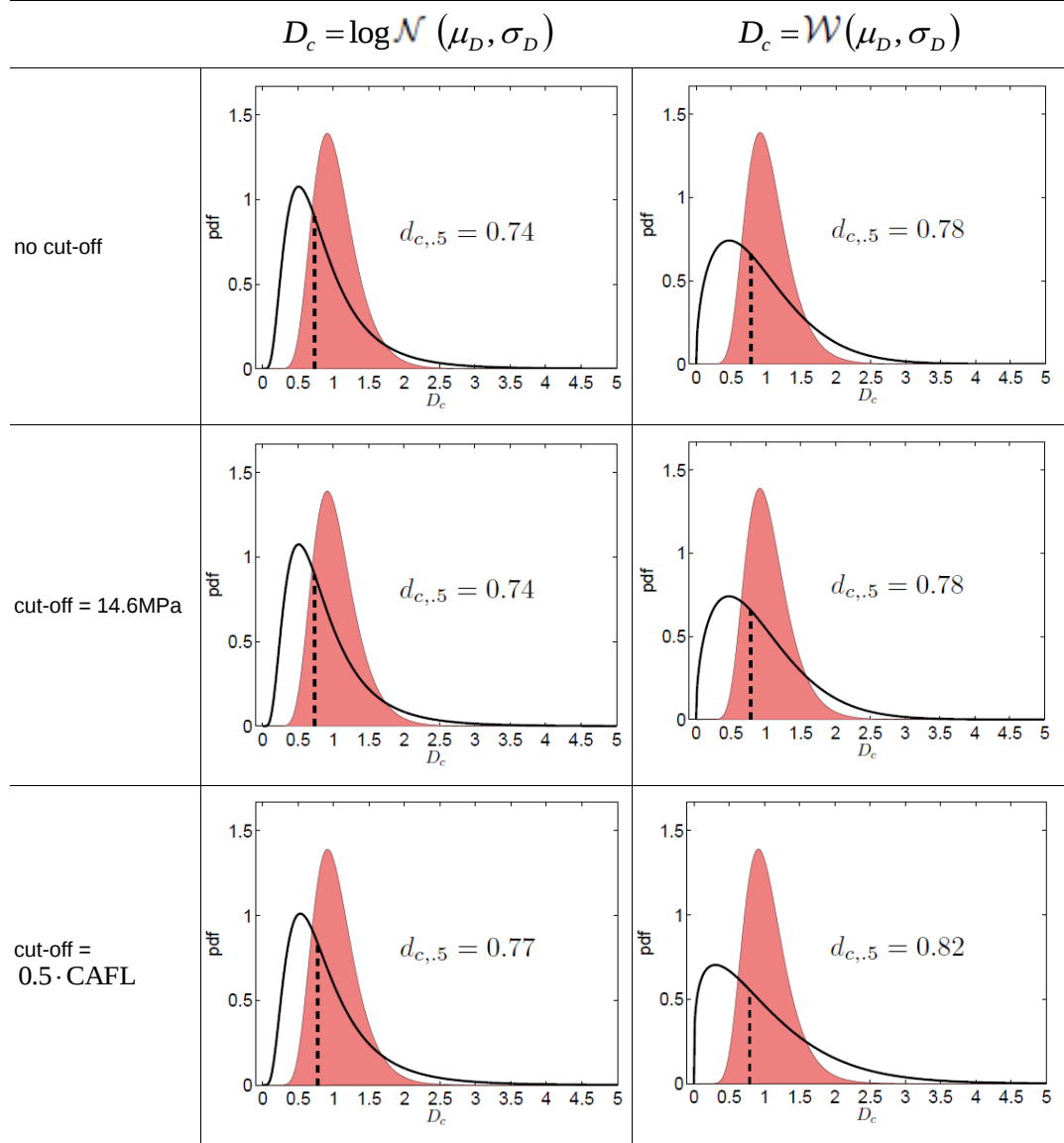


Fig. 44 shows the ML-MCS-based VA characteristic S-N curve for the cover-plate dataset. The combination no cut-off / $D_c = \mathcal{W}$, which gave the best ML fit of VA test results, was chosen. The slope m_2 is equal to -11.4 and the median value of the critical damage sum, $d_{c,.5}$, is equal to 1.31. The ML-MCS-based VA S-N curve was re-scaled according to Equation 38 for direct comparison with standard VA S-N curves; the comparison in Fig. 45 gives the following information:

- The AASHTO FAT40 S-N curve is over-conservative with respect to the ML-MCS S-N curve at all stress range levels;
- The EN3 FAT45 S-N curve is slightly over-conservative with respect to the ML-MCS S-N curve at stress ranges between 18 MPa (cut-off) and 21 MPa (CAFL), and at stress ranges higher than 25MPa;
- The IIW FAT50 and the EN3 FAT45 S-N curves are unsafe below 18MPa (cut-off).

Fig. 46 shows the ML-MCS-based VA characteristic S-N curve for the in-plane gusset data-set. The combination no cut-off / $D_c = \log N$, which gave the best ML fit of VA test results, was chosen. The slope m_2 is equal to -7.7 and the median value of the critical damage sum, $d_{c,5}$, is equal to 0.74. The comparison of ML-MCS based re-scaled VA S-N curve with standard VA S-N curves (see Fig. 47) gives following information:

- The AASHTO FAT56 S-N curve is considerably over-conservative with respect to the ML-MCS S-N curve at stress ranges between 15MPa (cut-off) and 27MPa;
- The EN3 FAT40 S-N curve is considerably over-conservative with respect to the ML-MCS S-N curve at stress ranges higher than 15MPa (cut-off);
- The AASHTO FAT56 and the EN3 FAT40 S-N curves are unsafe below 15MPa (cutoff);
- The IIW FAT50 S-N curve is unsafe at stress ranges lower than 17MPa (cut-off).

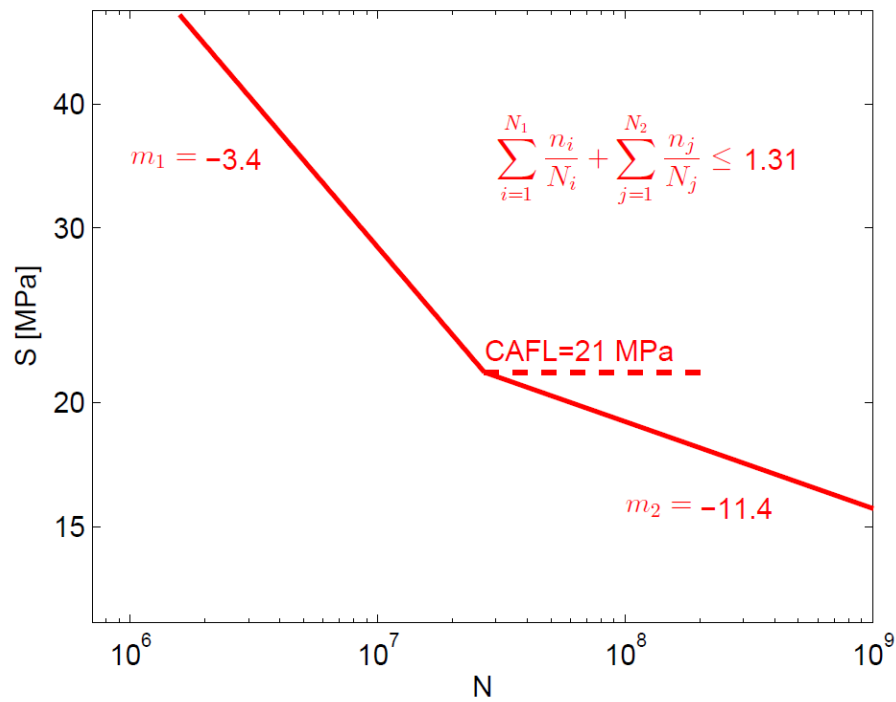


Fig. 44 Cover plate, MCS-ML characteristic VA S-N curve, no cut-off / $D_c = \mathcal{W}$.

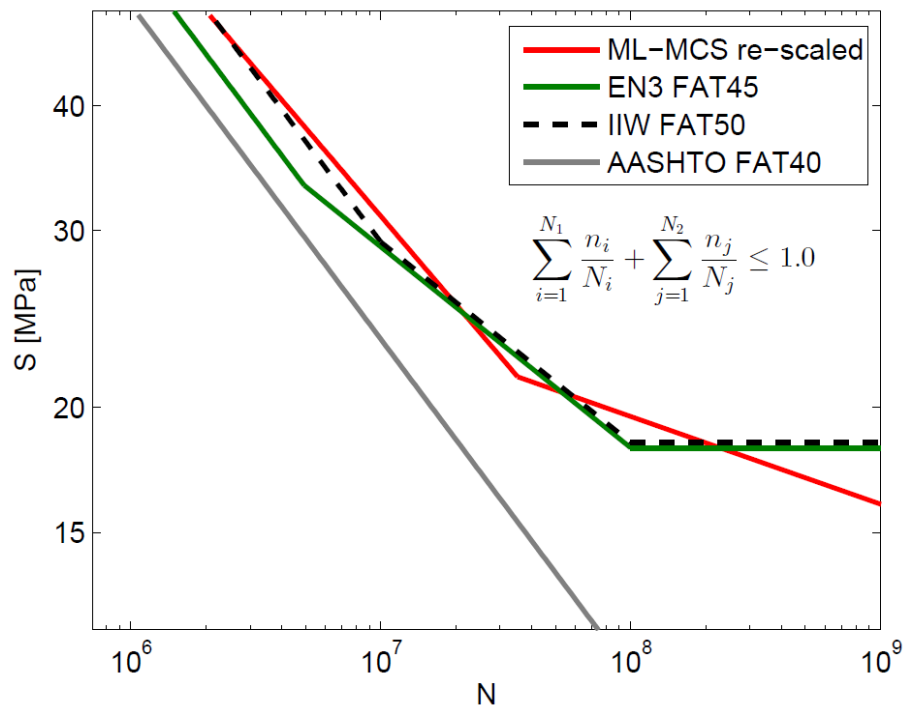


Fig. 45 Cover plate, curves comparison.

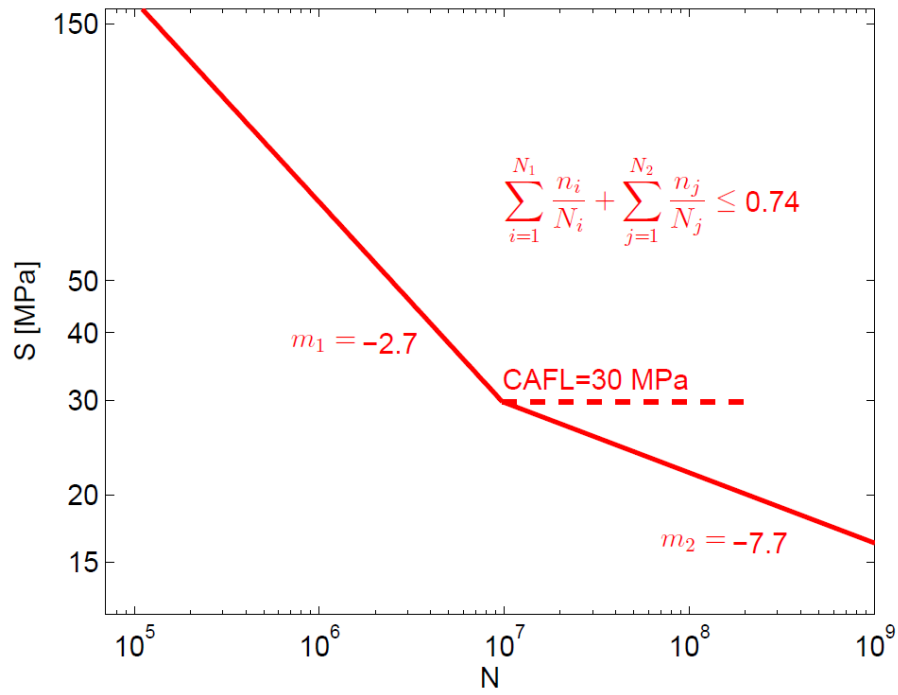


Fig. 46 In-plane gusset, MCS-ML characteristic VA S-N curve, no cut-off / $D_c = \log \mathcal{N}$.

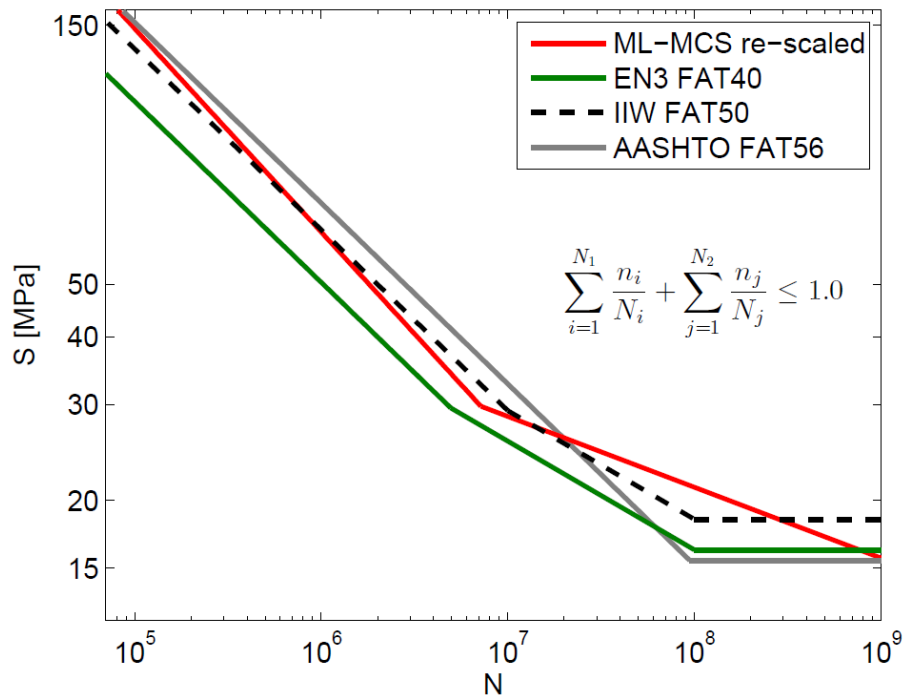


Fig. 47 In-plane gusset, comparison curves.

Tab. 12 Characteristic VA S-N curves

Cover plate				
	FAT [MPa]	CAFL [MPa]	m_1	m_2
ML-MCS	46	21.5	-3.4	-11.4
Eurocode	45	33.2	-3.0	-5.0
IIW	50	29.2	-3.0	-5.0
AASHTO	40	-	-3.0	-
In-plane gusset				
	FAT [MPa]	CAFL [MPa]	m_1	m_2
ML-MCS	54	29.8	-2.7	-7.7
Eurocode	40	29.5	-3.0	-5.0
IIW	50	29.2	-3.0	-5.0
AASHTO	56	-	-3.0	-

The ML-MCS-based S-N model provides the best fit of VA experimental points (both failures and run-outs); it is recalled here that each VA experimental point (σ_t, n_t) is transformed in damage sum, by mean of the VA S-N curve. In Fig. 45 and Fig. 47 the re-scaled ML-MCS characteristic VA S-N curves for cover-plate and in-plane gusset data-sets are compared to VA characteristic S-N curves from standards.

In order to offer to reader another simple way to compare the ML-MCS VA S-N model to VA S-N models from standards, the experimental failure points of both studied data-sets were transformed in damage sums by using ML-MCS characteristic VA S-N curve and characteristic S-N curves from standards (see Tab. 12). Results are compared in Fig. 48: white "bubbles" represent the damage sum at failure with the size of the bubble being proportional to the frequency of observation, and red (gray on black/white copy) circles represent the mean observed damage sum, $\bar{d}$.

Fig. 48 shows that ML-MCS and IIW models give the most realistic estimates of critical damage sum ($\bar{d} < 2.5$), while Eurocode and AASHTO models give over-conservative estimates of damage sum ($\bar{d} > 3.4$).

It is noted that the "bubble diagram" represents only a fast and simple way to compare different VA S-N models since run-out observations are not considered as well as the variability of VA S-N model parameters. In order to perform a rigorous comparison of different VA S-N models, the ML-MCS scheme presented in Section 4.2.1 should be used.

4.4.2 Disqualification of outliers

When observing failure damage sums for the cover-plate data-set (see first column of Tab. 13), it is evident that the lowest damage sum (third spectrum in Tab. 6, failure at $104 \cdot 10^6$ cycles) lies an abnormal distance from other damage sum values. In order to assess whether this value can be classified as an outlier, the box-plot method was applied to observed failure log-damage sums. Results of the analysis are shown in Tab. 13: since the lowest failure log-damage sum (-2.60) exceeds the lower outer fence (F_{lo}), it can be classified as extreme outlier. The cover-plate VA data-set was re-analysed by neglecting the failure point corresponding to the classified outlier: the ML estimates of parameters of D_c distribution and the ML estimate of slope change, $\Delta \hat{m}$ are shown in

Tab. 14. The new ML-MCS-based D_c distribution, with outlier disqualification, is compared to the original ML-MCS-based D_c distribution (without outlier disqualification), in Fig. 49.

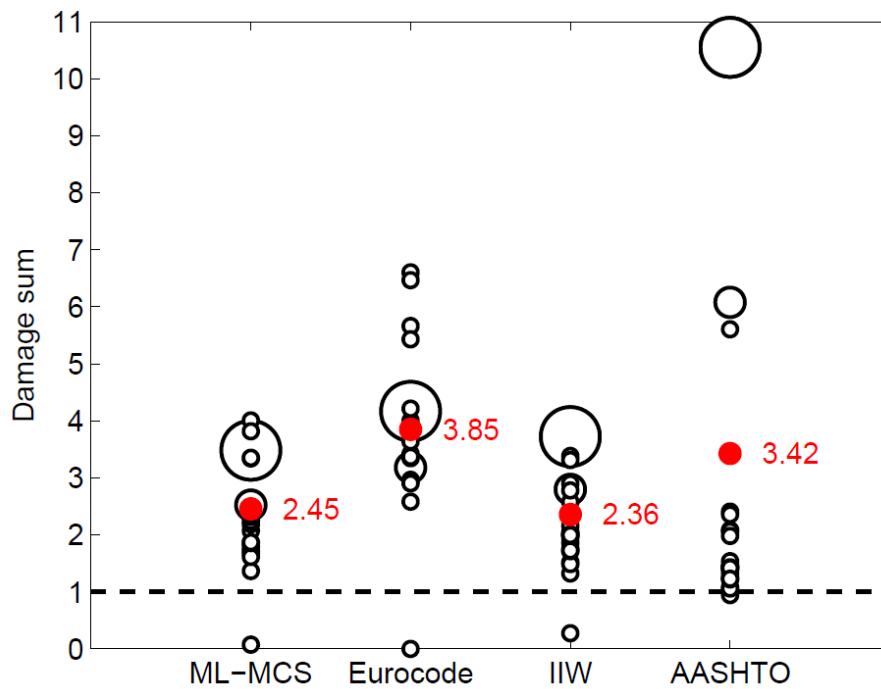
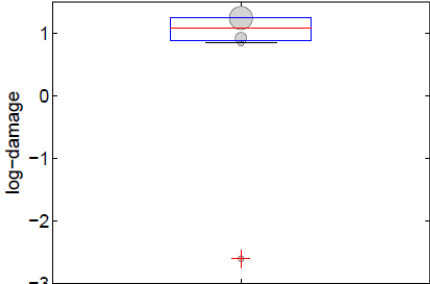


Fig. 48 Bubble diagram, characteristic damage sums.

Tab. 13 Cover plate, outlier disqualification

$\log(D)$	Quartiles	Fences	
1.25	$Q_3 = 1.25$ $Q_1 = 0.88$ $IQ = 0.36$	$F_{uo} = 2.34$ $F_{ui} = 1.79$ $F_{li} = 0.34$ $F_{lo} = -0.21$	
1.25			
1.25			
1.25			
-2.60			
0.92			
0.92			
0.84			

Tab. 14 Cover plate, outlier disqualification: models comparison

Outlier disqualification	Cut-off case	D_c	$\hat{\mu}_D$	$\hat{\sigma}_D$	$\Delta\hat{m}$	$\overline{n\hat{L}L}$
no	no cut-off	$\mathcal{W}$	1.70	1.40	8	1183
yes	no cut-off	$\log \mathcal{N}$	0.27	0.58	6	934

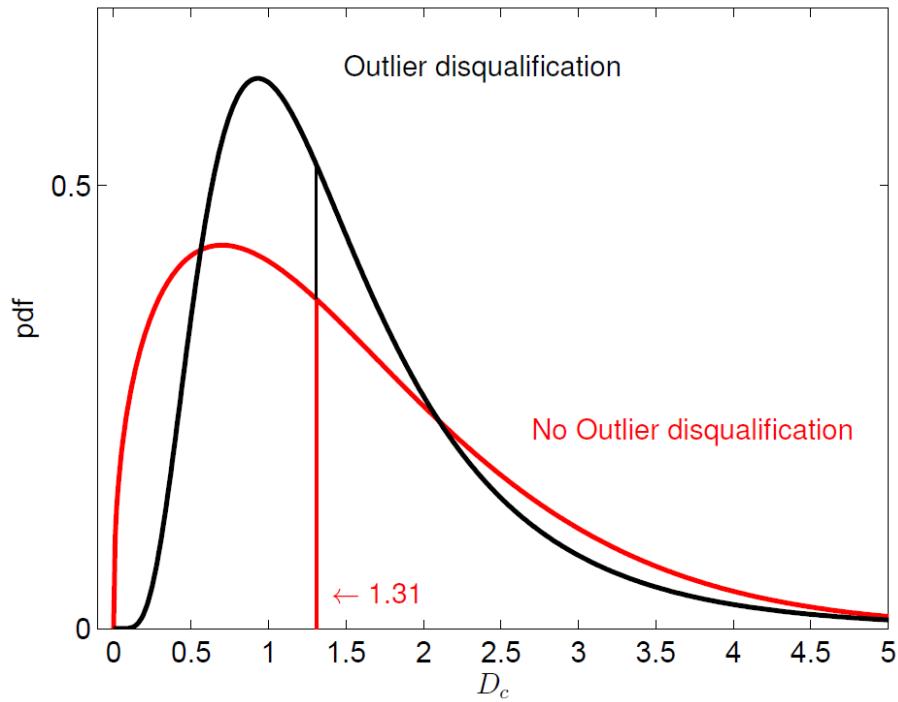


Fig. 49 Cover plate, D_c distribution with outlier disqualification.

4.5 Conclusions and discussion

In this Chapter a new approach was proposed for estimation of VA S-N models of components. The new approach has general applicability. It allows to estimate both the VA S-N curve and the critical value of damage sum, which are used in Miner's equation, when the fatigue life of a welded joint under the effect of VA loading is computed. The novel contribution of this approach consists in re-simulating VA experimental fatigue tests and in estimating the S-N curve second slope, m_2 , and the critical damage sum, D_c , with ML method. The new approach was implemented in the Matlab Toolbox TB1 (see Appendix III) and applied to two study cases of welded joints.

The conclusions from this Chapter are as follows:

- There are two critical aspects which are related to the use of Miner's rule in fatigue assessment of welded components under VA loadings: 1) The choice of the S-N curve slope, m_2 , below the CAFL; and 2) The choice of the critical value of damage sum, D_c . The ML-MCS approach presented here allows to estimate the S-N curve second slope, m_2 , and the critical damage sum, D_c , on the basis of experimental VA results (including both failures and run-outs), by using the ML-MCS CA S-N stochastic model and without need for any arbitrary assumption;
- The estimates of VA S-N stochastic model for both cover plate and in-plane gusset data-sets (see Tab. 8 and Tab. 9) show that the VA S-N curve without stress range cut-off maximizes the Likelihood information: the cases of cut-off at 14.6MPa (lowest Eurocode FAT detail) and cut-off at $0.5 \cdot \text{CAFL}$ are not supported by VA experimental results. This indicates that try-outs with lower cut-off values should be tried. Indeed, Baptista found experimentally on in-plane gusset detail a cut-off limit at $\approx 0.25 \text{ FAT}$ [11];
- Density plots and probability plots (see Fig. 29 to Fig. 39) support the hypothesis of asymptotic normality of estimators $\hat{\mu}_D$; $\hat{\sigma}_D$ (parameters of critical damage distribution, D_c) and of $n\hat{LL}$ (sampled negative Log-Likelihood), for both cover plate and in-plane gusset data-sets;

- For the cover plate data-set with outlier disqualification, the graph $\overline{n\hat{L}L} - \Delta m$ shows that $D_c = \log \mathcal{N}$ with $\Delta \hat{m} = 6$ gives the best fit in terms of Likelihood (see Tab. 14). The disqualification of the outlier point does not affect the median value of D_c distribution (which remains equal to 1.31). For the in-plane gusset data-set, the graph $\overline{n\hat{L}L} - \Delta m$ shows that $D_c = \log \mathcal{N}$ with $\Delta \hat{m} = 5$ gives the best fit in terms of Likelihood (see Fig. 43). This confirms the existence of a second slope m_2 (in disagreement with results from [28],[52],[88],[6]) but the influence of the loading characteristics on the second slope m_2 cannot be assessed. These values are higher than Haibach proposition (taken in the Eurocode EN1993-1-9) of $\Delta m = 2$;
- The VA S-N model correlation matrix shows that there is no linear correlation between the parameters of the S-N curve and the parameters of the D_c distribution, for both cover plate and in-plane gusset data-sets (see Fig. 26 and Fig. 27). This seems to indicate that the location and the scale parameter of D_c are influenced only by the characteristics of loading history and not by the fatigue strength of the studied detail. Further analysis of different fatigue details is needed to confirm this indication;
- For the cover plate data-set, where experimental results were obtained under wideband Rayleigh type stress range spectra having short block lengths, the median value of D_c is equal to 1.31, while for the in-plane gusset study case, where experimental results were obtained under 3-blocks spectra with stress ranges cycling down from constant maximum stress (no over-loads which could retardate the crack opening), the median value of D_c is equal to 0.74. This seems to indicate that there is a strong influence of loading characteristics and in particular of load sequence on the critical damage sum distribution, which is coherent with results from [88] and [34];
- Re-scaling of ML-MCS characteristic VA S-N curve at $D_c = 1$ allowed for direct comparison of ML-MCS-based VA S-N curves with standard VA S-N curves, showing that S-N curve from AASHTO specification is highly over-conservative for the cover plate data-set at all stress ranges (see Fig. 45), and that all standard S-N curves (Eurocode, IIW, AASHTO) are slightly over-conservative at stress ranges lower than 30 MPa, for the in-plane gusset data-set (see Fig. 47);
- ML fit provides support $D_c = \log \mathcal{N}$ for both the cover-plate data-set and for the in-plane gusset data-set, once disqualification of one extreme outlier of the cover-plate data-set is made ;
- The "bubble" diagram, representing the characteristic damage sums for both considered data-sets, confirm that the ML-MCS approach gives the most realistic estimates of critical damage sum, together with IIW (see Fig. 48).

The findings listed above suggest that the ML-MCS approach defined in this Chapter constitutes a powerful tool for re-definition of second slope, m_2 , and of critical damage sum, D_c . The proposed approach was applied on two welded details VA experimental data-sets and validated by using diagnostic plots, Likelihood plots and "bubble" plot.

5 Fatigue reliability assessment of existing bridges

5.1 Introduction

In fatigue life assessment of steel bridges the reliable consideration of extreme loads which lie near the CAFL is of primary importance. In order to accomplish this requirement, three problems need to be adequately solved:

1. Reliable definition of the CAFL and of its variability;
2. Reliable definition of the damage accumulation rule, especially for stress range cycles below the CAFL;
3. Reliable consideration of extreme loads which lie near the CAFL.

The ML-MCS approach, presented in Chapters 3 and 4, provides a solution to problems 1) and 2).

Weigh-In-Motion (WIM) technology provides a powerful tool to face problem 3). Prior to recent years, probabilistic approaches were used to extrapolate short period of recorded data to long return periods (i.e. one recorded week extrapolated to 120 years) [9],[19]; however recent improvements in WIM technology allows to use recorded traffic data of one or more years.¹⁵

Chen et al. [17] proposed a new reliability framework for estimation of fatigue life of suspension bridges under the long-term effect of railway, highway and wind loading. The limit state function was defined by using the daily sum of m-power stress ranges, which were computed by using probabilistic model of railway, highway and wind loading; WIM measurements were used to characterize highway traffic loading history. The fatigue failure probability at fatigue critical locations was based on the critical fatigue damage accumulation event; the CAFL exceedance event was not included in the analysis. The critical value of the damage sum, D_c , was modeled by $\log \mathcal{N}(0, 0.3)$ distribution, and S-N curves were defined according to British standards [81]. The framework was applied on the Tsing Ma suspension bridge in Hong Kong.

Guo et al. [33] developed a new approach, integrating WIM data and probabilistic finite element (FE) analysis, for fatigue assessment of steel bridges. The limit state function was based on the critical damage accumulation event, while the CAFL exceedance event was not included in the analysis. S-N curves were defined according to AASHTO bridge design specifications [4] and D_c was modeled by $\log \mathcal{N}(0, 0.3)$ distribution. The approach was applied on the Throgs Neck suspension bridge, connecting the Throgs Neck section of the Bronx in New York city with the Bay Terrace section of Queens.

In both approaches, adopting conventional assumption for definition of S-N curves and of D_c and modeling the CAFL as deterministic value, may yield inaccurate insight into reliability index values and undo the beneficial effect of accurate modeling of the traffic loads as well as of the bridge structure.

D'Angelo et al. [22] provided a fatigue-life reliability assessment of the Venoge highway bridge, on the highway A1 in Switzerland. WIM measurement-based traffic simulation was used to get realistic traffic loading history. The fatigue failure probability at two critical locations was based on the CAFL exceedance event, where the random CAFL was defined according to EN1993-1-9 [2]. Since the analysis was based only on the CAFL exceedance event, the results could be largely over-conservative. Again, adopting

¹⁵ M. Treacy has shown in his PhD thesis [82] that at least 6 months of record are needed to reliably extrapolate extreme traffic value

conventional assumption for the definition of S-N curves may yield inaccurate insight into reliability index values.

Sudret et al [78],[32] proposed a probabilistic approach for fatigue life assessment of structures subjected to thermal fatigue. This approach incorporated all kinds of uncertainties affecting the thermal fatigue behavior such as those related to the fatigue strength, to the thermo-mechanical model and to the thermal loading. The proposed probabilistic framework provides an effective tool for computing probability of fatigue failure as function of the expected design life and for estimating the importance of random variables in response sensitivity. Nevertheless, this approach has following limitations when applied to the case of fatigue assessment of road bridges: 1) The critical value of the damage sum at failure, D_c is assumed to be equal to 1.0. This assumption does not allow to take into account the deviation effects related to loading history; 2) The accumulated damage is computed with a one-slope S-N curve having an asymptotic endurance limit, S_D ; the critical point related to the influence on the fatigue damage of stress range cycles below the CAFL, which is of primary importance in mechanical fatigue evaluation of welded joints, is not addressed; 3) In modelling the random number of cycles to failure ($\ln(N) = \lambda(S) + \sigma(S)\xi(\omega)$, where $\xi(\omega)$ is a standard Normal random variable), the aleatory uncertainty of the fatigue limit is not taken into account: the ML-MCS approach presented in this study shows that the consideration of aleatory uncertainty of the CAFL is essential for computing quantiles of fatigue life (see Section 3.3.3).

In this Chapter a novel framework for fatigue reliability assessment of steel bridges under the long-term effect of traffic loading, is presented. WIM measurements are used to characterize traffic loading history. The fatigue reliability framework takes in account both the CAFL exceedance event and the critical damage accumulation event: failure probability and related reliability index are computed by estimating the joint failure probability of these two events. The S-N curves and the critical damage sum, D_c , are defined according to the ML-MCS approach, which has been presented in Chapters 3 and 4.

The reliability framework which is presented in this Chapter was implemented in the Matlab Toolbox TB2 (see Appendix III). The Matlab Toolbox TB2 was validated using the structural reliability analysis software STRUREL [67].

The Chapter is organized as follows:

- In Section 5.2 the new framework for fatigue reliability analysis is presented;
- In Section 5.3 the fatigue reliability analysis framework is applied to the Venoge bridge study case;
- In Section 5.4 results of the fatigue reliability analysis of the Venoge bridge are presented and compared to Eurocode EN1993-1-9 [2];
- In Section 5.5 results are discussed.

5.2 New framework for fatigue reliability analysis

In this Section the framework for the fatigue reliability analysis of road bridges under the long-term effect of traffic loading, is presented.

Welded joints in road bridges are subjected to VA fatigue loadings; hence fatigue life assessment is done by using linear damage accumulation rule (Miner's rule [58]):

$$D_d = \sum_i \frac{n_i}{\exp(Y(m_0, m_1, \sigma, V; S_i))} + \sum_j \frac{n_j}{\exp(Y(m_0, m_1, m_2, \sigma, V; S_j))} \leq D_c \quad (\text{Eq. 39})$$

where:

- D_d is the total accumulated damage;
- V is the natural logarithm of the CAFL;
- n_i is the i -cycle of the loading spectrum, which has nominal stress range S_i above the CAFL;
- n_j is the j -cycle of the loading spectrum, which has the nominal stress range S_j below the CAFL;
- m_1 is the slope of the S-N curve for a nominal stress range above the CAFL;
- $m_2 = m_1 - \Delta m$ is the slope of the S-N curve for a nominal stress range below the CAFL and it allows to take in account the gradual exceedance of the propagation threshold by lower stress ranges as the crack length increases;
- m_0 is the log-intercept of S-N curve having a slope equal to m_1 ;
- D_c is the critical value of the accumulated damage.

The variables listed above are defined in the S-N model.

The validity of Eq. 39 is conditioned to the fact that there is at least one cycle of the loading spectrum which exceeds the CAFL:

$$S_{max} \geq \exp(V) \quad (\text{Eq. 40})$$

where S_{max} is the maximum stress range of the loading spectrum.

Therefore the assessment of the fatigue failure probability of considered critical details asks for consideration of two events:

- E_1 : Critical damage accumulation
- E_2 : CAFL exceedance

To have fatigue failure, both events E_1 and E_2 must happen. From the definition of conditional probability:

$$P_f = P(E_1 \cap E_2) = P(E_1|E_2)P(E_2) = P(E_1^*)P(E_2) \quad (\text{Eq. 41})$$

where $E_1^* = E_1|E_2$ represents critical damage accumulation conditioned to CAFL exceedance; definition of limit state equations related to E_1^* and E_2 is discussed below.

Event E_1^* : critical damage accumulation conditioned to CAFL exceedance

Following from Equation 40, the limit state equation related to the conditioned event E_1^* may be written as:

$$g_1(X_1) = D_c - \sum_i \frac{n_i}{\exp(Y(m_0, m_1, \sigma, V(\mu_V, \sigma_V); S_i))} - \sum_j \frac{n_j}{\exp(Y(m_0, m_1, \sigma, V(\mu_V, \sigma_V), m_2; S_j))} \quad (\text{Eq. 42})$$

The failure event E_1^* may be described in the following way:

$$E_1^* = \{g_1(\underline{X}_1) \leq 0\} \quad (\text{Eq. 43})$$

where $\underline{X}_1$ is the vector of basic random variables $(Y(m_0, m_1, m_2, \sigma, V(\mu_V, \sigma_V)), D_c(\mu_D, \sigma_D))$. It is noted that both the epistemic uncertainty of model parameters and the aleatory uncertainty of the CAFL, of the fatigue life and of the critical damage sum, are taken into account. Having defined the failure event E_1^* , the probability of failure $P(E_1^*) = P(E_1 \cap E_2) / P(E_2)$ may be defined as follows:

$$P(E_1^*) = \int_{g_1(\underline{X}_1) \leq 0} f_{\underline{X}_1}(\underline{x}_1) d\underline{x}_1 \quad (\text{Eq. 44})$$

where $f_{\underline{X}_1}(\underline{x}_1)$ is the pdf of $\underline{X}_1$. The related reliability index is:

$$\beta_1 = -\Phi^{-1}(P(E_1^*)) \quad (\text{Eq. 45})$$

Since the limit state function $g_1(\underline{X}_1)$ is non linear in the basic random variables $\underline{X}_1$, the β_1 index has to be computed using linearization approaches (First order reliability method, FORM, or Second order reliability method, SORM) or MCS technique.

Event E_2 : CAFL exceedance

The limit state equation related to the event E_2 is defined as follows:

$$g_2(\underline{X}_2) = \exp(V) - z_p \quad (\text{Eq. 46})$$

where $\underline{X}_2$ is the vector of basic random variables $(z_p, V(\mu_V, \sigma_V))$, with z_p indicating the p – return level of maximum stress range.

It is noted that both the epistemic uncertainty of S-N model parameters (μ_V, σ_V) and the aleatory uncertainty of the CAFL are taken into account.

In analogy with β_1 , the reliability index related to the event E_2 is:

$$\beta_2 = -\Phi^{-1}(P(E_2)) = -\Phi^{-1} \left(\int_{g_2(\underline{x}_2) \leq 0} f_{\underline{X}_2}(\underline{x}_2) d\underline{x}_2 \right) \quad (\text{Eq. 47})$$

where $f_{\underline{X}_2}(\underline{x}_2)$ is the pdf of $\underline{X}_2$. The reliability problem may be solved again using linearization approaches or MCS technique.

5.3 Study case

The framework presented in Section 5.2 was applied to the Venoge bridge, located on the A1 Swiss motorway between Lausanne and Geneva (see Fig. 50). The Venoge bridge consists of two identical independent bridges (one for each direction) with a 219 meter length divided in four spans (see Fig. 51). Each bridge was constructed in 1961 with two girders and then enlarged in 1995 to four girders (see Fig. 52). Due to the slow lane (heavy traffic) location only the enlarged bridge section is considered in this study (see Fig. 52).

One shall notice that this bridge has no fatigue problems. In order to make the study more interesting, it was assumed that a cover-plate existed on the 1995 enlargement as it exists on the 1961 bridge, see Fig. 53 (but which today does not support the heavy traffic lane). In this fictitious case, the welded cover plate on the left lower flange, at the first mid span (Section *A* in Fig. 51), was identified as the fatigue critical location. Two year-traffic induced stress history was generated using WIM measurements and a 3-dimensional finite element (FE) model of the bridge. The stress response influence line at the fatigue critical location was characterized with a refined three-dimensional (3-D) FE model which takes into account the effective transverse load repartition, the cracking of the concrete slab and the elastic stiffness of stud connectors; the FE model was calibrated and validated using experimental load-vs-strain results from bridge static tests [16]. MSC/Nastran [59] was used for the analysis. For the sake of brevity the reader is referred to [19] for complete information on the calibration of FE model of the bridge.

The influence line for the nominal stress S_n , at the critical section *A*, is shown in Fig. 54. The bridge fatigue failure probability, during the 100-year design life, was computed by solving the reliability problem of Equation 41 at the critical location.



Fig. 50 The Venoge bridge, Ecublens (Switzerland).

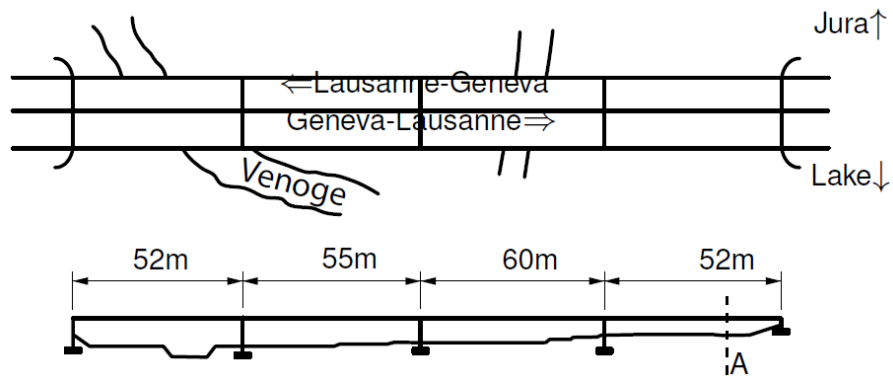


Fig. 51 In-plane view and elevation of the Venoge bridge.

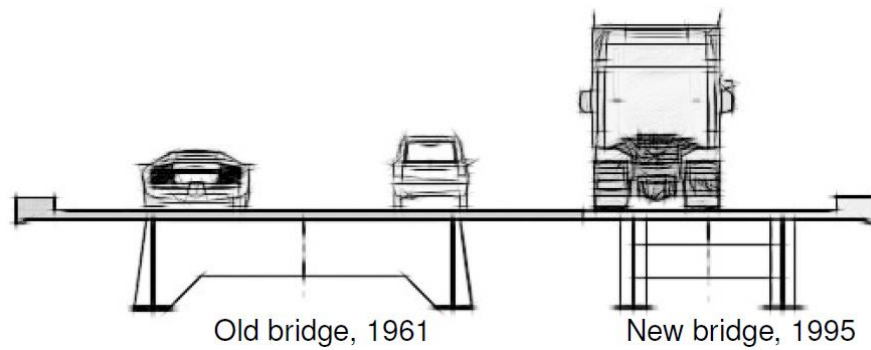


Fig. 52 Lane configuration on the Venoge bridge.



Fig. 53 The critical detail: welded cover plate (located on the 1961 bridge).

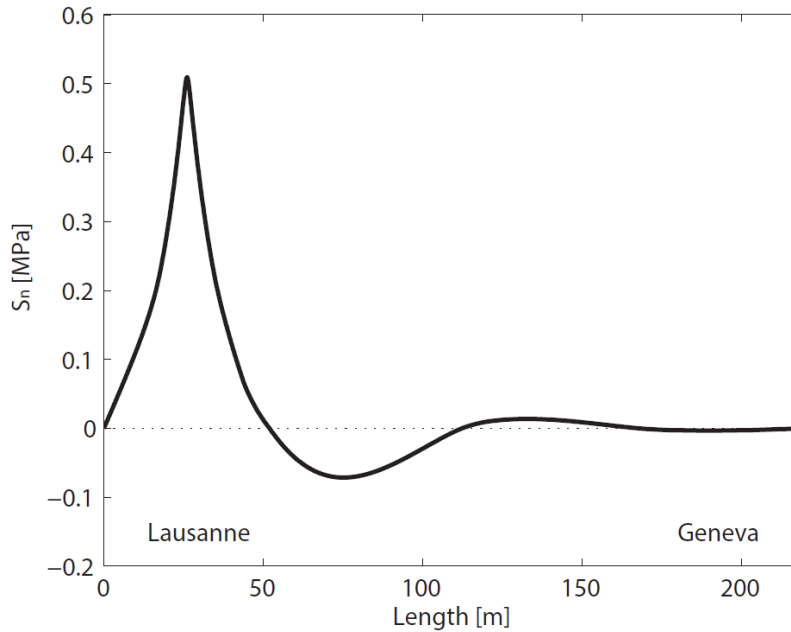


Fig. 54 Influence line for nominal stress S_n at the critical section, 1 ton crossing axle.

5.3.1 Traffic analysis

WIM recorded data of the Denges station (only 1km away from the bridge) over a 2 year period (1.01.2006 - 31.12.2007) were considered in order to reconstruct the time-history of bridge crossing heavy vehicles. Direction Lausanne-Geneva was analysed due to the unavailability of complete data for the opposite direction. WIM devices can capture static vehicle axle weights, which allow for characterization of traffic demand in terms of:

- time of passage
- vehicle speed
- number of axles
- gross total weight
- axles weight
- inter-axles distance

A code which reads recorded WIM data, classifies vehicles according to GR03-EUR13 classification [55] and produces the 2-year history of bridge crossing was implemented in the module TrafficAnalysis of Matlab Toolbox TB2 (see Appendix III). Only heavy vehicles with gross total weight (GTW) higher than 10 tons were considered herein and in fatigue calculation because lighter vehicles give a negligible contribution to the bridge fatigue damage [72].

Fig. 55 shows that there is not a significant evolution of heavy traffic from the observed period (2006-2007) to year 2013, both in terms of observed/classified vehicles and of observed crossing axles. Fig. 56 shows the comparison between the empirical pdf of GTWs for the observed period 2006-2007, and the empirical pdf of GTWs for the year 2013, confirming that there is not significant evolution from the observed period (2006-2007) to year 2013. As presented in the report Part 1 : traffic, simulations can be used to study in more detail the influence of traffic evolution, generate pdf of GTWs, and perform fatigue reliability analyses.

For the considered study case, the influence length of the critical detail is higher than 52m: the load dynamic amplification factor, ϕ_d , is set equal to 1.0, according to Brühwiler et al. [15] which recommended to use $\phi_d = 1.0$ for influence lengths longer than 40m.

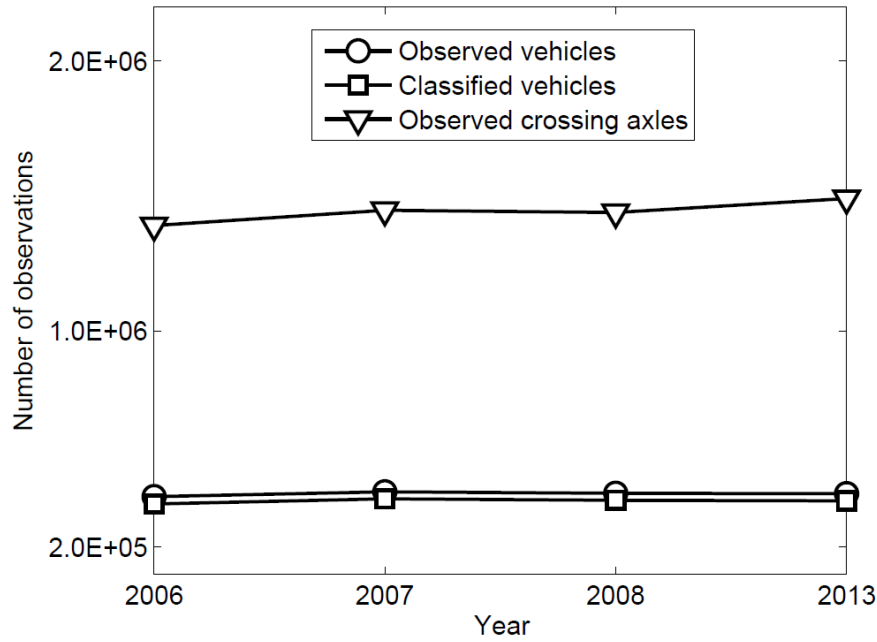


Fig. 55 Heavy traffic observations (GTW > 10 tons).

5.3.2 Fatigue resistance models

Two different S-N models were considered for the fatigue reliability analysis of the Venoge bridge:

1. ML-MCS-based VA S-N model for the cover-plate data-set with outlier disqualification, defined in Section 4.4;
2. Eurocode FAT45 S-N curve, with conventional hypothesis for critical damage D_c .

In the ML-MCS-based S-N model, the VA S-N curve is represented by a multivariate Normal vector, whose parameters are defined in Tab. 8 and Fig. 26, while the critical value of damage sum, D_c , is a Log-Normal random variable, whose parameters are defined in Tab. 14.

In the Eurocode-based S-N model, the slopes and the CAFL of the CA S-N curve are deterministic values and the only source of fatigue scatter is represented by the Normal random variable m_0 , which is defined according to [80]¹⁶; the critical value of damage sum, D_c , is a log-Normal random variable with mean equal to 0 and standard deviation equal to 0.3, according to JCSS PMC [43].

¹⁶ The location and the scale parameters of the m_0 Normal distribution are computed as follows:

$$\mu_{m_0} = \ln(\text{FAT}_{med}) = \ln\left(10^{\log_{10}(\text{FAT}) + 2 \cdot s_R}\right), \text{ with } s_R = 0.033$$

$$\sigma_{m_0} = 0.5 \cdot \left(\mu_{m_0} - \ln(2 \cdot 10^6) - 3 \cdot \ln(\text{FAT}) \right)$$

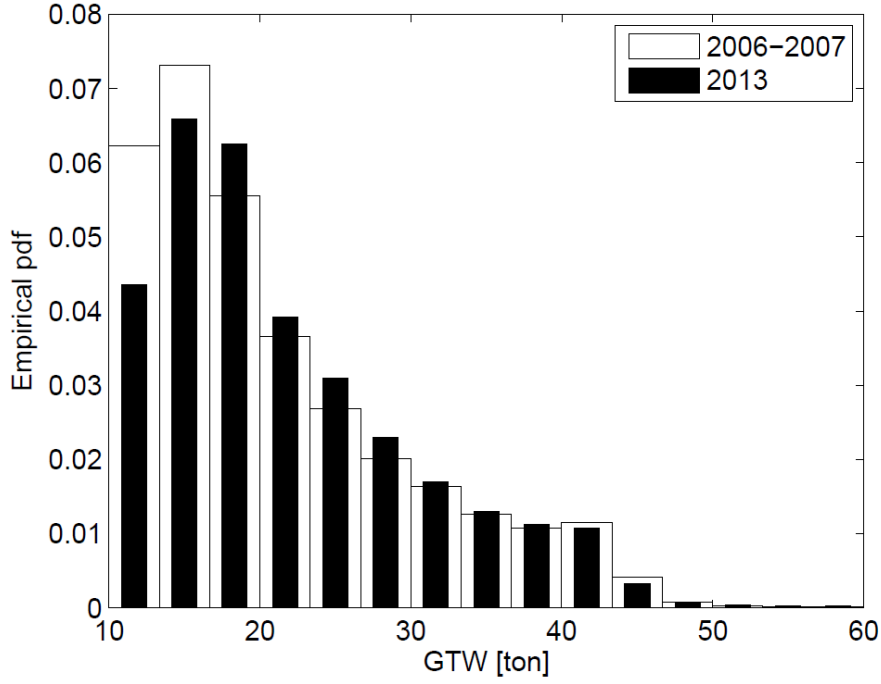


Fig. 56 Empirical pdf of observed yearly GTWs.

Values of S-N curve parameters and of D_c parameters are resumed in Fig. 57.

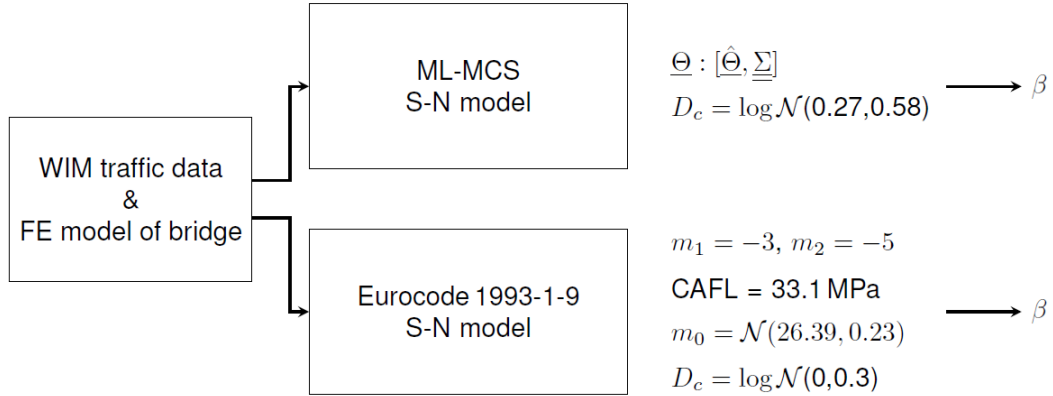


Fig. 57 Scheme for calculation of β reliability index.

5.3.3 Resolution of reliability problem $P(E_1^*)$

5.3.3.1 ML-MCS S-N model

The limit state equation related to the event E_1^* has been formulated in Equation 42.

The S-N model variables $(m_0, m_1, m_2, \sigma, V(\mu_v, \sigma_v)), D_c(\mu_D, \sigma_D)$ are defined in the ML-MCS VA S-N stochastic model for cover plate data-set with outlier disqualification (see Tab. 8, Fig. 26 and Tab. 14).

In order to compute the weekly loading spectra, $(S_i, n_i)_{w|w=1, \dots, 104}$, the following scheme was followed:

1. The stress history at the critical section, $S_n(t)$, was obtained by using the crossing axle time history, $A(t)$, and the influence line shown in Fig. 54;
2. The 2-year stress history $S_n(t)$ was divided in 104 weekly blocks, $S_{n,w}(t)|_{w=1, \dots, 104}$
3. From each block the stress range spectrum $(S_i, n_i)_{w=1, \dots, 104}$ was computed using a rain-flow algorithm [26].

This scheme has been implemented in the module `TrafficAnalysis` of Matlab Toolbox TB2 (see Appendix III).

The probability of failure $P(E_1^*)$ was computed by using MCS method:

$$P(E_1^*) = \int_{g_1(\underline{X}_1) \leq 0} f_{\underline{X}_1}(\underline{x}_1) d\underline{x}_1 = \frac{\eta_F}{\eta} \quad (\text{Eq. 48})$$

where η is the number of simulations performed and η_F is the number of failures¹⁷. During each simulation, one value of the limit state function $g_1(\underline{X}_1)$ (see Equation 42) is computed, by sampling a value of the multivariate Normal vector, $(m_0, m_1, m_2, \sigma, \mu_V, \sigma_V, \mu_D, \sigma_D)$ ¹⁸, and by taking $n_y \cdot 52$ stress range spectra, $(S_i, n_i)_w$, where n_y is the design life in years.

For high values of β the proposed MCS scheme would require a number of simulations too high (the estimation of $\beta = 6$ requires 10^{11} simulations): in this case the limit state function g_1 is re-written as follows:

$$g_1(\underline{X}_1) = D_c - D_d \quad (\text{Eq. 49})$$

and the probability of failure $P(E_1^*)$ is computed with FORM, after having characterized the random variable D_d with MCS (using 10^5 simulations¹⁹).

The MCS resolution scheme proposed in this section has been implemented in the `E1definition` and `E1betacalc` modules of Matlab Toolbox TB2 (see Appendix III).

¹⁷ If MCS is asked to estimate a probability in the order of 10^{-y} , then approximately 10^{y+2} simulations are necessary to achieve an estimate with a coefficient of variance in the order of 10%.

¹⁸ At each simulation a different S-N curve and a different value of critical damage sum is considered, by taking into account the aleatory uncertainty of the S-N curve, of the CAFL and of the critical damage sum. The epistemic uncertainty of model parameters is also taken into account in repeated sampling of the multivariate Normal vector.

¹⁹ It was observed that 10^6 simulations give the same values of D_d distribution parameters as with 10^5 simulations.

5.3.3.2 Eurocode S-N model

As discussed in Section 5.3.2, in the Eurocode EN1993-1-9 S-N model, the only source of the fatigue scatter is represented by the normal random variable m_0 . In this case the simplified approach proposed by Chen et al. [17] is used for computing $P(E_1^*)$; the approach is presented below.

The 104 weekly loading spectra $(S_i, n_i)_{w|w=1, \dots, 104}$ were computed by using the same scheme as previously presented for the ML-MCS S-N model. Then the weekly sum of m-power stress ranges was computed as:

$$r_{w_k} = \sum_i n_i (s_i)^{-m_1} + \frac{1}{0.74 \text{FAT}^{m_1-m_2}} \sum_j n_j (s_j)^{-m_2} \quad \text{for } (k = 1, \dots, 104) \quad (\text{Eq. 50})$$

Within the assumption that the weekly sum of m-power stress ranges is a Normal random variable with location parameter μ_{R_w} and scale parameter σ_{R_w} , the observed sample r_{w_k} was used to make inference about the distribution parameters. Under the assumption of no-traffic growth, the sum of m-power stress ranges over n_y year design life is:

$$R_d = \sum_{k=1}^{n_y \cdot 52} R_{w_k} \quad (\text{Eq. 51})$$

Location parameter μ_{R_d} and scale parameter σ_{R_d} were computed by making use of the central limit theorem [84]:

$$\begin{aligned} \mu_{R_d} &= n_y \cdot 52 \cdot \mu_{R_w} \\ \sigma_{R_d} &= \sqrt{n_y \cdot 52} \cdot \sigma_{R_w} \end{aligned} \quad (\text{Eq. 52})$$

The limit state equation g_1 was then re-written in function of the R_d random variable:

$$g_1(R_d, m_0, D_c) = D_c - \frac{R_d}{\exp(m_0)} = D_c - D_d \quad (\text{Eq. 53})$$

Since g_1 is a non-linear function of Normal random variables R_d, m_0, D_c , the probability of failure $P(E_1^*)$ was computed by using FORM with Hasofer-Lind approach [39].

5.3.4 Resolution of reliability problem P(E₂)

The limit state equation related to the event E_2 has been formulated in Equation 46.

In order to characterize the random variable z_p (return level of maximum stress range), the stress history was analysed by following the classical extreme value (EV) theory approach.

5.3.4.1 EV theory approach for estimation of maximum stress range return levels z_p

The observed stress history was divided in 104 weekly blocks. Each weekly stress history block was transformed in a stress range weekly spectrum by using the rain-flow algorithm.

From each weekly spectrum the maximum stress range, $s_{\max, w}$, was extracted.

Within the assumption of constant pattern of variation, the observed $s_{\max, w}$ can be modeled as independent observations from a GEV family distribution; the cdf of the GEV family distribution is defined as follows:

$$G(z) = \exp \left\{ - \left[1 + \xi \left(\frac{z - \tau}{\phi} \right) \right]^{\frac{-1}{\xi}} \right\} = Pr\{S_{max} \leq z\} \quad (\text{Eq. 54})$$

where τ is the location parameter, ϕ is the scale parameter, and ξ is the shape parameter.

The estimates of the extreme quantiles of the weekly maximum stress ranges were computed by inverting the cdf of the GEV family distribution (see Equation 54):

$$z_p = G(z)^{-1} = \begin{cases} \tau - \frac{\phi}{\xi} \left\{ 1 - [-\ln(1-p)]^{-\xi} \right\} & \text{for } \xi \neq 0 \\ \tau - \phi \ln \{-\ln(1-p)\} & \text{for } \xi = 0 \end{cases} \quad (\text{Eq. 55})$$

with z_p representing the return level for the stress range, which is exceeded one time every p weeks. The plot $z_p - p$ is called return level plot.

The pdf of the GEV family distribution is defined as follows:

$$g(z) = \frac{1}{\phi} t(z)^{\xi+1} \exp(-t(z)) \quad (\text{Eq. 56})$$

where

$$t(z) = \begin{cases} \left(1 + \xi \frac{z - \tau}{\phi} \right)^{\frac{-1}{\xi}} & \text{for } \xi \neq 0 \\ \exp \left(-\frac{z - \mu}{\phi} \right) & \text{for } \xi = 0 \end{cases}$$

Under the assumption that observed maximum stress ranges are independent random variables which follow a GEV distribution, the negative log-Likelihood function is defined as the sum of $-\ln(g(z_i))$ over the 104 week observed period.

From Equation 56 it follows:

$$nLL(\tau, \phi, \xi) = 104 \ln(\phi) + \left(1 + \frac{1}{\xi} \right) \sum_{i=1}^{104} \ln \left[1 + \xi \left(\frac{z_i - \tau}{\phi} \right) \right] + \sum_{i=1}^{104} \left[1 + \xi \left(\frac{z_i - \tau}{\phi} \right) \right]^{\frac{-1}{\xi}}, \quad \text{for } \xi \neq 0 \quad (\text{Eq. 57})$$

provided that $1 + \xi \left(\frac{z_i - \tau}{\phi} \right) > 0$, for $i = 1 \dots 104$.

Or:

$$nLL(\tau, \phi, \xi) = 104 \ln(\phi) + \sum_{i=1}^{104} \frac{z_i - \tau}{\phi} + \sum_{i=1}^{104} \exp \left[- \left(\frac{z_i - \tau}{\phi} \right) \right], \quad \text{for } \xi = 0 \quad (\text{Eq. 58})$$

The ML estimate of GEV model parameters, $(\hat{\tau}, \hat{\phi}, \hat{\xi})$, was obtained by minimizing the negative log-Likelihood function (see Equations 57 and 58); since no analytical solution is available a numerical solution is needed for minimizing the negative log-Likelihood function.

In most cases $\xi > -0.5$ (the case $\xi \leq -0.5$ is rarely encountered since it corresponds to distributions with a very short bounded upper tail) and the vector $(\hat{\tau}, \hat{\phi}, \hat{\xi})$ follows a multivariate normal distribution with mean equal to (τ, ϕ, ξ) and covariance matrix equal to the inverse of the observed Fisher information matrix, $I((\tau, \phi, \xi))$ [23].

The assumption of approximate normality of ML estimators was verified and the variance of the return level, $\text{Var}(z_p)$, was computed using the Δ method:

$$\text{Var}(z_p) = \left[\frac{\partial z_p}{\partial \tau}, \frac{\partial z_p}{\partial \phi}, \frac{\partial z_p}{\partial \xi} \right]^T \cdot \underline{\underline{\text{Cov}}}(\tau, \phi, \xi) \cdot \left[\frac{\partial z_p}{\partial \tau}, \frac{\partial z_p}{\partial \phi}, \frac{\partial z_p}{\partial \xi} \right] \quad (\text{Eq. 59})$$

where $\underline{\underline{\text{Cov}}}(\tau, \phi, \xi)$ is the covariance matrix of the vector $(\hat{\tau}, \hat{\phi}, \hat{\xi})$.

The validity of the extrapolation of stress range return levels, z_p , can be assessed using following diagnostic plots:

- Probability plot, which compares the empirical cumulative distribution function, $G^e(z_i) = i/(104+1)$, and the fitted distribution function, $G(z_i)$ (see Equation 54); deviation from diagonal indicates inaccuracy of the GEV fitting. Since for increasing i both $G^e(z_i)$ and $G(z_i)$ tend to unity, the probability plot is not very useful for large values of p , which is the zone of greatest interest in extreme value assessment problems;
- Quantile plot, which consists of points $\left[\left(G^{-1}(i/(104+1)) \right), z_i \right], i = 1, \dots, 104$; deviation from the diagonal indicates inaccuracy of the GEV fitting;
- Comparison of the pdf of the fitted GEV distribution with histogram of data. This plot is less useful than the probability and the quantile plots since the form of histogram varies with the choice of intervals.

It has to be noticed that the approach presented above is valid if:

1. The series $s_{\max, w}$ is stationary: this assumption is realistic for the Venoge study case and can be verified by checking the diagram of observed maximum stress ranges;
2. The series $s_{\max, w}$ is time-independent: this hypothesis is unrealistic for practical applications. Therefore in practical applications, provided that long-range

dependence at extreme levels is weak, then $s_{\max, w}$ follow the same distributional limit laws as those of independent series [18]. This means that $s_{\max, w}$ may be fitted with GEV family distribution, provided that long-range dependence is eliminated; the GEV model parameters themselves are different from those that would have been obtained by considering an independent series, but since the parameters of the GEV family distribution have to be estimated anyway, this is not relevant.

ML estimation of GEV parameters, validation of z_p extrapolation by means of diagnostic plots and computation of $\text{Var}(z_p)$ were implemented in the module E2definition of the Matlab Toolbox TB2 (see Appendix III).

5.3.4.2 ML-MCS S-N model

The random variable $V(\mu_V, \sigma_V)$ was defined in the ML-MCS VA S-N stochastic model for cover plate data-set with outlier disqualification (see Tab. 8 and Fig. 26).

Once the random variables V and z_p of the limit state equation g_2 (see Equation 46) were defined, the probability of failure $P(E_2)$ was computed with FORM. The FORM resolution scheme for computation of $P(E_2)$ was implemented in the module E2betacalc of Matlab Toolbox TB2 (see Appendix III).

5.3.4.3 Eurocode S-N model

According to the Eurocode EN1993-1-9 S-N model, the CAFL is a deterministic value, which is determined by cutting the characteristic S-N curve at $5 \cdot 10^6$ cycles. In order to perform a reliability analysis on the failure event E_2 , the CAFL of the median S-N curve, $\exp(\mu_V)$, was computed by assuming a fatigue log-life standard deviation equal to 0.033 [80] (as for the computation of parameters of m_0 , see Fig. 57) and the standard deviation, σ_V , was determined by assuming a two-standard deviation difference between the mean and the characteristic value of the CAFL distribution:

$$V = \mathcal{N}(3.65, 0.08) \quad (\text{Eq. 60})$$

The probability of failure $P(E_2)$ was computed again with FORM.

5.4 Results

5.4.1 Event E_1^* : Critical damage accumulation

5.4.1.1 Eurocode S-N model

Observed weekly sum of m-power stress ranges was computed according to Eq. 50. Fitted normal distribution are plotted in Fig. 58. The probability plot of observed weekly sums is shown in Fig. 59.

The parameters of the Normal distribution R_w are:

$$\begin{aligned}\hat{\mu}_{R_w} &= 2.35 \cdot 10^6 \\ \hat{\sigma}_{R_w} &= 6.01 \cdot 10^5\end{aligned}\quad (\text{Eq. 61})$$

The parameters of the Normal distribution R_d , over the design life, are:

$$\begin{aligned}\hat{\mu}_{R_d} &= 1.22 \cdot 10^8 \cdot n_y \\ \hat{\sigma}_{R_d} &= 4.33 \cdot 10^6 \cdot \sqrt{n_y}\end{aligned}\quad (\text{Eq. 62})$$

for $n_y = 1, \dots, 100$.

The coefficient of variation of R_w is equal to 0.256, while the coefficient of variation of R_d ranges from 0.256 (at beginning of design life) to 0.003 (at the end of design life). At the end of design life almost all randomness of accumulated damage is considered in the variable m_0 (see Equation 53).

After having defined all random variables in the limit state function g_1 (see Equation 53), the reliability index $\beta_1 = -\Phi^{-1}(P(E_1^*))$ was computed over the design life of the bridge, by using FORM (see Tab. 15). The results obtained by using FORM were checked by using SORM (see again Tab. 15): the difference in β_1 computation is negligible and use of SORM is not justified.

It is recalled here that $P(E_1^*)$ represents the probability of having critical damage accumulation (E_1), given that CAFL has been exceeded (E_2).

Tab. 15 Calculation of β_1 : Eurocode S-N model

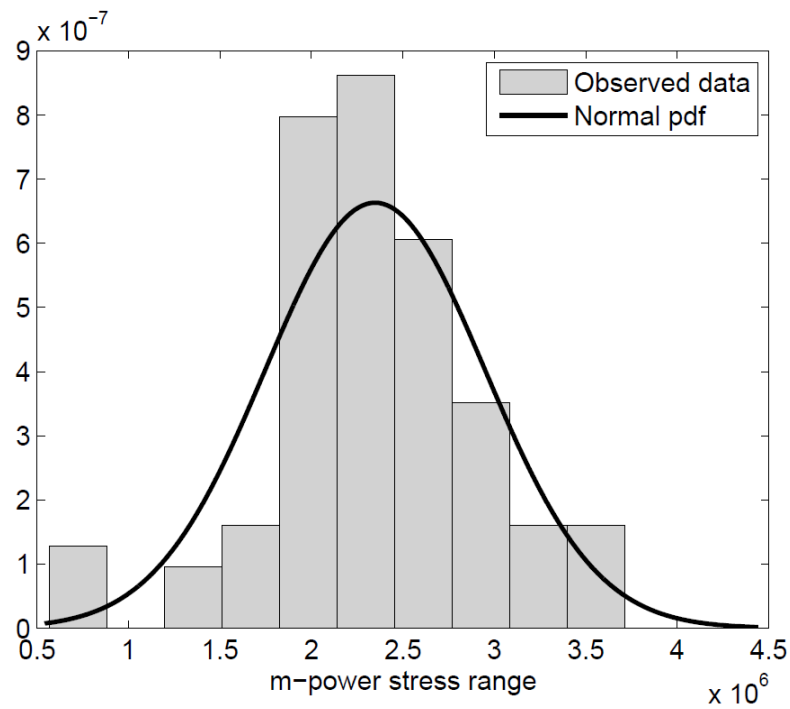
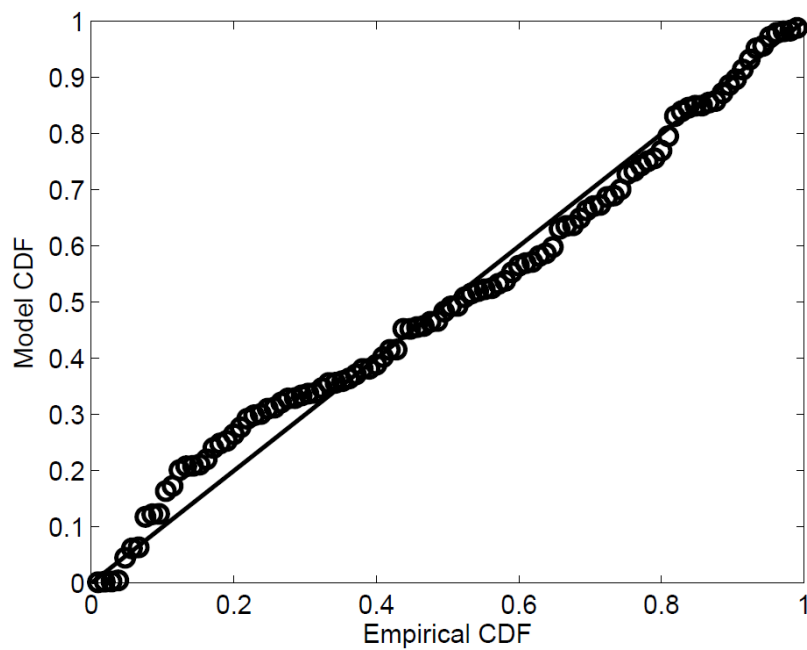
Years	FORM	SORM	Difference
25	12.028	12.030	< 0.01%
50	10.196	10.190	< 0.01%
75	9.124	9.122	< 0.01%
100	8.363	8.361	< 0.01%

5.4.1.2 ML-MCS S-N model

The 104 weekly stress range spectra were generated by following the scheme presented in Section 5.3.3 (ML-MCS S-N model). The reliability index $\beta_1 = -\Phi^{-1}(P(E_1^*))$ was computed over the design life of the bridge, by using MCS (see Equation 48). Values of ML-MCSbased reliability index β_1 are compared to values of Eurocode-based reliability index β_1 in Tab. 16. Note that all values are largely above the target reliability index specified by the codes, see section 5.4.3.

Tab. 16 Calculation of β_1 : Eurocode vs ML-MCS

Years	Eurocode	ML-MCS
25	12.03	11.03
50	10.20	9.86
75	9.12	9.17
100	8.36	8.68

**Fig. 58** Observed weekly sum of m -power stress ranges.**Fig. 59** Probability plot of observed weekly sum of m -power stress ranges.

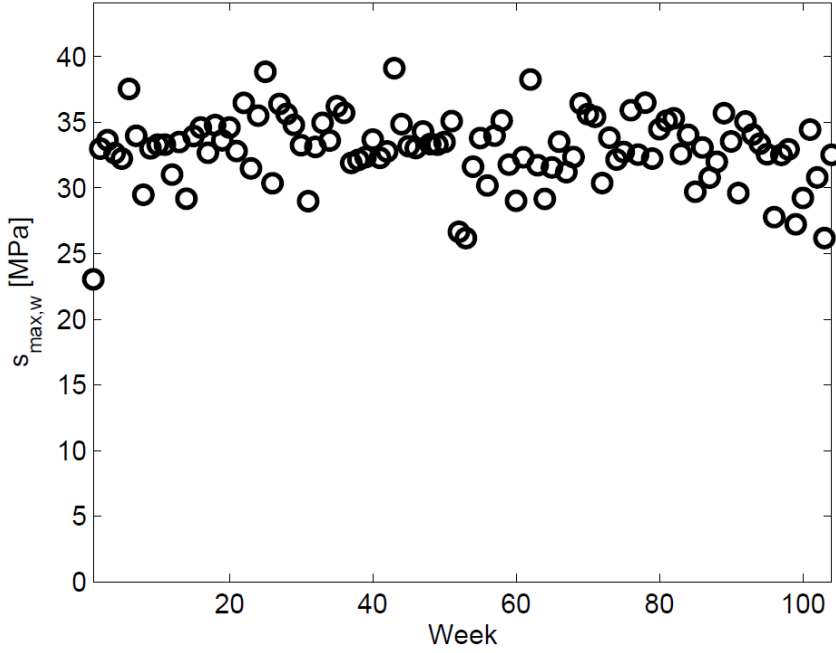


Fig. 60 Observed weekly maximum stress ranges.

5.4.2 Event E₂: CAFL exceedance

The observed weekly maximum stress ranges, $s_{\max, w}$, over the period 2006-2007, are shown in Fig. 60: the assumption of constant pattern of variation, which allows to model the observed $s_{\max, w}$ as independent observations from a GEV family distribution, is supported.

Fit of GEV distributions family to observed $s_{\max, w}$ leads to the ML estimate:

$$(\hat{\mu}, \hat{\sigma}, \hat{\xi}) = (32.08, 2.88, -0.37) \quad (\text{Eq. 63})$$

$$\underline{\underline{\text{Cov}}}(\hat{\mu}, \hat{\sigma}, \hat{\xi}) = \begin{bmatrix} 1.7 \cdot 10^{-3} & 5.3 \cdot 10^{-3} & 4.3 \cdot 10^{-3} \\ \dots & 4.1 \cdot 10^{-2} & -1.2 \cdot 10^{-2} \\ \dots & \dots & 9.2 \cdot 10^{-2} \end{bmatrix} \quad (\text{Eq. 64})$$

Since the shape factor, ξ , is higher than -0.5, the hypothesis of asymptotic normality of ML estimators is supported.

Fig. 61 represents $\hat{z}_p$ against observed values (empirical return levels). The probability and quantile plots for assessing the accuracy of the GEV model fitted to the observed data are shown in Fig. 62 and Fig. 63. After having defined z_p random variable in limit state function g_2 (see Equation 46), the reliability index $\beta_2 = -\Phi^{-1}(P(E_2))$ was computed over the design life of the bridge, by using FORM for both Eurocode S-N model and ML-MCS S-N model.

Computed values of reliability index β_2 are presented in Tab. 17.

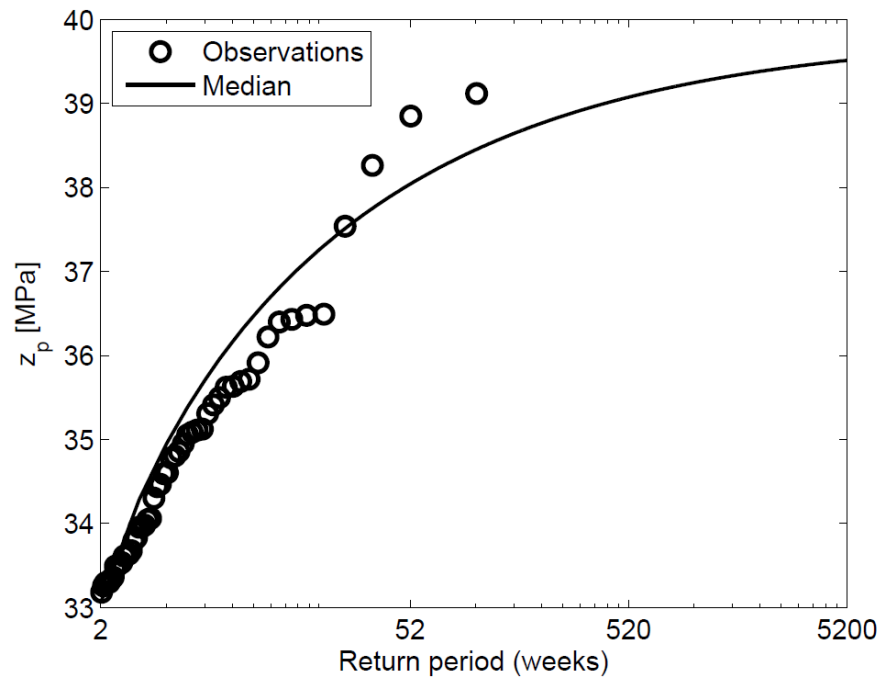


Fig. 61 Return level plot of the weekly maximum stress range.

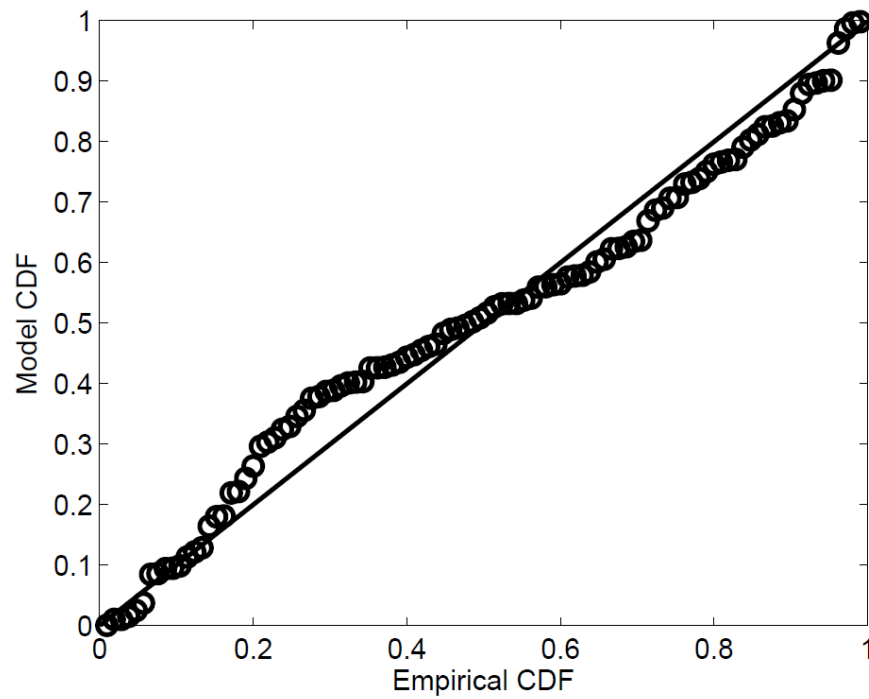


Fig. 62 Probability plot of z_p .

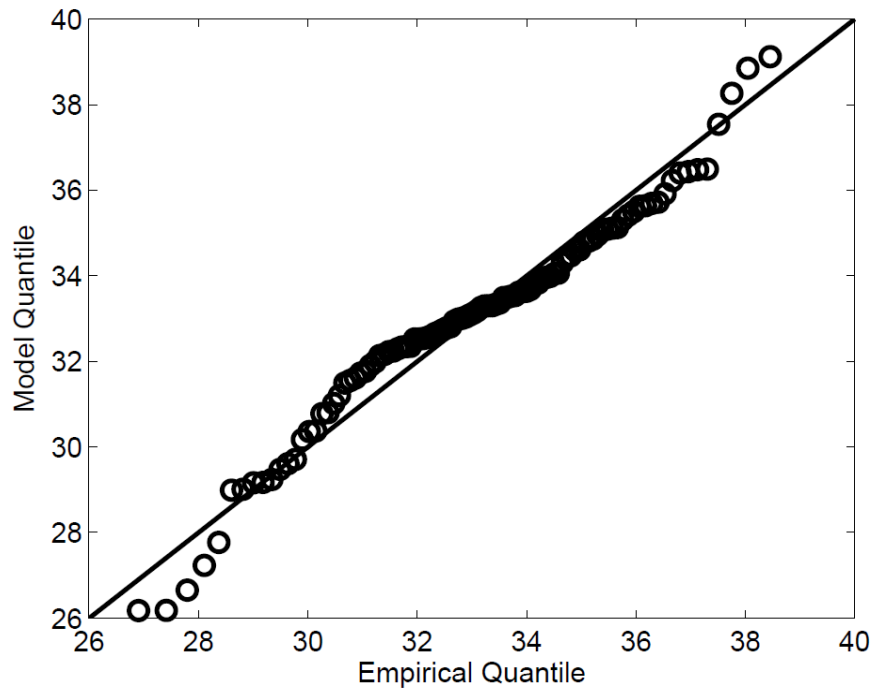


Fig. 63 Quantile plot of z_p .

Tab. 17 Calculation of β_2 : Eurocode vs ML-MCS

Years	Eurocode	ML-MCS
25	-0.15	-0.94
50	-0.16	-0.94
75	-0.17	-0.94
100	-0.17	-0.94

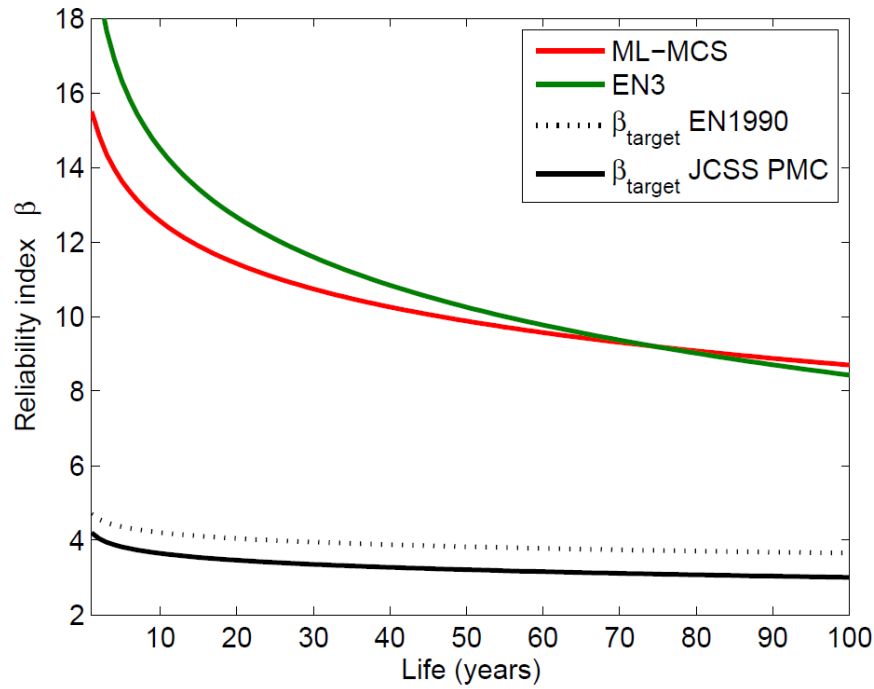


Fig. 64 Evolution of reliability index β over the design life.

5.4.3 Event $E_1 \cap E_2$: fatigue failure

According to Equation 41, the probability of fatigue failure is given by the product of probabilities $P(E_1^*)$ and $P(E_2)$; the fatigue reliability index $\beta = -\Phi^{-1}(P(E_1^* \cdot E_2))$ was computed using results of Sections 5.4.1 and 5.4.2.

The evolution of the fatigue reliability index β over the 100 year design life is shown in Fig. 64, both for the Eurocode S-N model and the ML-MCS S-N model. Following reliability indexes are also plotted in Fig. 64 for direct comparison:

- β_t EN1990, based on an annual failure probability equal to $1.3 \cdot 10^{-6}$, corresponding to a β annual equal to 4.7 (Reliability class RC2, assuming independency, [1] Annex C, Chapter C6);
- β_t JCSS, based on an annual failure probability equal to $1.3 \cdot 10^{-5}$, corresponding to a β annual equal to 4.2 (Normal relative cost of safety measure and moderate consequence of failure, assuming independency [43]).

As already mentioned, the reliability index values for this bridge and detail are largely above the specified target values from the codes. Since the detail studied is fictitious and unfavorable, one can generalize and write that the fatigue safety of this bridge is very high. The main reason for it is the low actual traffic volume compared to the design traffic. Indeed, extrapolating the evolution of the reliability index and target values show that the fatigue life is above 1'000 years (both for EN1990 and JCSS, it would be wrong to try to compute a precise value at this point). Thus, referring to this scenario as scenario 0, two virtual scenarios of future traffic growths and their effects on reliability, damage and fatigue life are now presented.

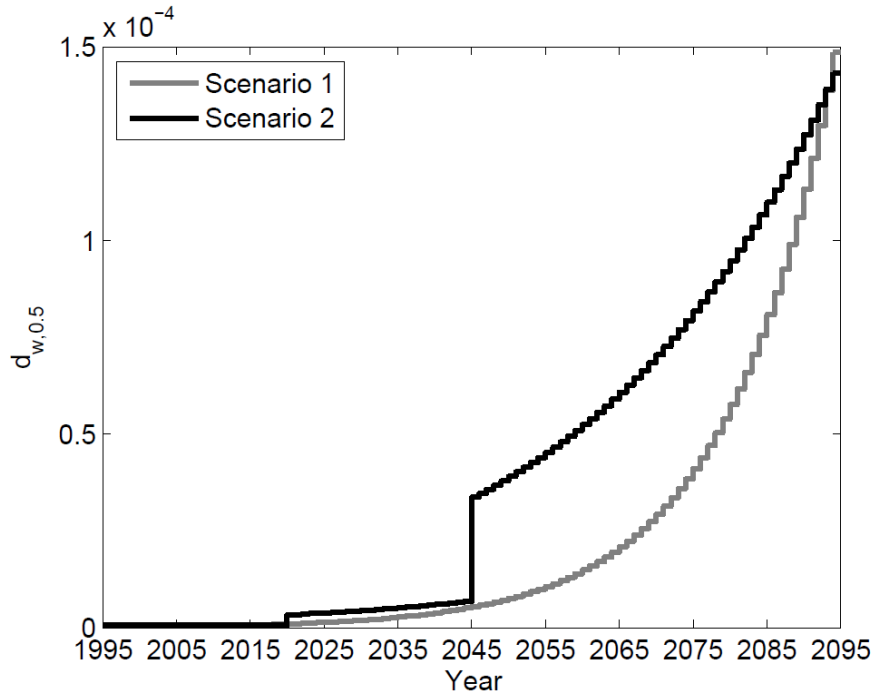


Fig. 65 Weekly damage growth: scenario 1) continuous 7% yearly increase, 2) continuous 3% increase as well as two discontinuous increments after 25 and 50 years.

Virtual traffic growth scenarios

In order to show the impact of future traffic evolution on the fatigue analysis, the effect of increasing the weekly accumulated fatigue damage was studied. This effect could be caused by, for example : an important and steady increase in the traffic volume due to attractiveness for this road axis, closing of another road axis, as well as changes in repartition of vehicle classes. Two very aggressive virtual scenarios were considered (see Fig. 65) for sake of demonstration of lowering the reliability index below the target value before the end of the bridge service life:

1. Weekly accumulated damage, D_w , having a 7% yearly increment of the median value, $d_{w,0.5}$ which corresponds to an annual traffic volume increase of 7%
2. Weekly accumulated damage, D_w , having a 500% discontinuous increment after 25 and 50 years of the median value, $d_{w,0.5}$, as well as a 3% yearly increment of the median value, $d_{w,0.5}$ (i.e. 3% annual traffic volume increase) starting from the first discontinuous increment.

Furthermore, for both scenarios, it was assumed that the weekly accumulated damage, D_w , is constant for the first 20 years (from 1995 to now).

It is noted that the considered virtual traffic evolution scenarios take into account only an increase in number of vehicles and not in truck loads; this cause a change in $P(E_1^*)$, while $P(E_2)$ is not affected by the traffic evolution scenarios. The choice of considering only a change in $P(E_1^*)$ is due to the fact that, for the considered Venoge study case, the ML-MCS-based $P(E_2)$ is constantly higher than 80% over the bridge design life (see Tab. 17) and a change in $P(E_2)$ would not have a significant effect on the global probability of failure $P(E_1^*) \cdot P(E_2)$.

Tab. 18 shows all computed reliability indexes β and provides a comparison with target reliability indexes from Eurocode EN1990 [1] and JCSS PMC [43]. Indeed, the values of the reliability index fall below the target values.

Fig. 66 shows the evolution of the reliability index β over the 100 year-design life for the two traffic growth scenarios, based on the ML-MCS S-N model. The histograms and the density maps of sampled $(d_d; d_c)$ in the ML-MCS S-N model-based β calculation are shown in Fig. 67, for no-traffic growth, and in Fig. 68, for the traffic growth scenario 2: the median of sampled d_d values increases from 0.007 (no traffic growth) to 0.4 (traffic growth), logically resulting in a significant reduction of β_1 . This reduction can be observed also in Fig. 69 and Fig. 70, where β_1 is proportional to the distance from the median of sampled $(d_d; d_c)$ to the limit state function (border between failure domain and no failure domain). The reduction of β_1 causes a decrease of β , that can be observed in Tab. 18.

Fig. 66 reveals the importance of the choice of target reliability index:

- For the traffic growth scenario 1, the ML-MCS S-N model-based β becomes lower than β_t EN1990 after 92 years, while it becomes lower than β_t JCSS after 97 years;
- For the traffic growth scenario 2, the ML-MCS S-N model-based β becomes lower than β_t EN1990 after 73 years, while it becomes lower than β_t JCSS after 81 years.

For these very aggressive traffic scenarios, the year triggering bridge maintenance is thus shifted by a minimum of 5 to 8 years (more in case of less aggressive scenarios) in function of the target reliability index chosen.

From the two points above it follows that the limit date, at which an inspection of the critical detail would be recommended, is postponed by 5 years for the traffic growth scenario 1 and by 8 years for the traffic growth scenario 2, when adopting JCSS PMC-based target reliability index instead of the Eurocode EN1990-based target reliability index. Note that both shifts would be larger in case of less aggressive traffic evolution scenarios.

For this case study, 2006-2007 WIM data were used and the reliability analysis is based on stationary load sequences; however any additional WIM data-set can be used to update estimated model and to take in account possible future traffic growth (non-stationary loading sequences): in this case probability distributions having time-dependent parameters $(D_d(\mu_D(t), \sigma_D(t)), z_p(\tau(t), \phi(t), \xi(t)))$ have to be used for reliability analysis.

As a final notice, one should mention that the reliability index values are probably somewhat too high, It has been observed that the repetition of simulations over short time periods (one week) leads to obtain, because of the central limit theorem, a normal distribution with a low covariance on traffic action effects, which then leads to overestimations of the reliability index. We advice whenever possible to use traffic simulations over longer periods (six months or one year).

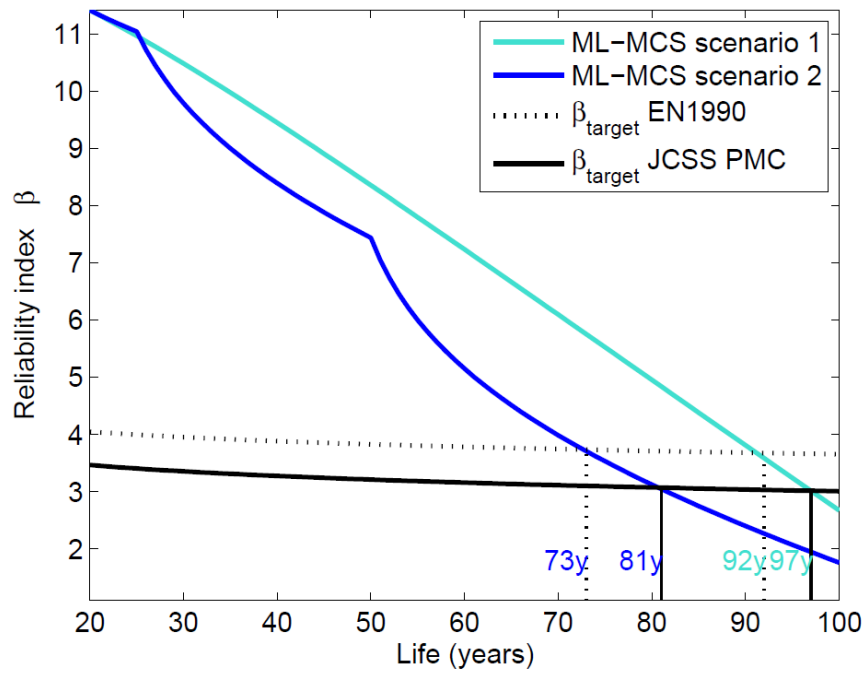


Fig. 66 Evolution of reliability index β over the design life, considering traffic growth.

Tab. 18 Calculation of β reliability index

S-N model	25 y.	50 y.	75 y.	100 y.
EN 1993-1-9	12.03	10.20	9.12	8.36
ML-MCS [†]	11.03	9.86	9.17	8.68
ML-MCS [†] traffic growth 1	10.97	8.36	5.53	2.67
ML-MCS [†] traffic growth 2	11.35	7.44	3.53	1.76
Eurocode EN1990	3.99	3.83	3.73	3.65
JCSS, PMC	3.40	3.21	3.09	3.00

[†] S-N model with disqualification of outliers

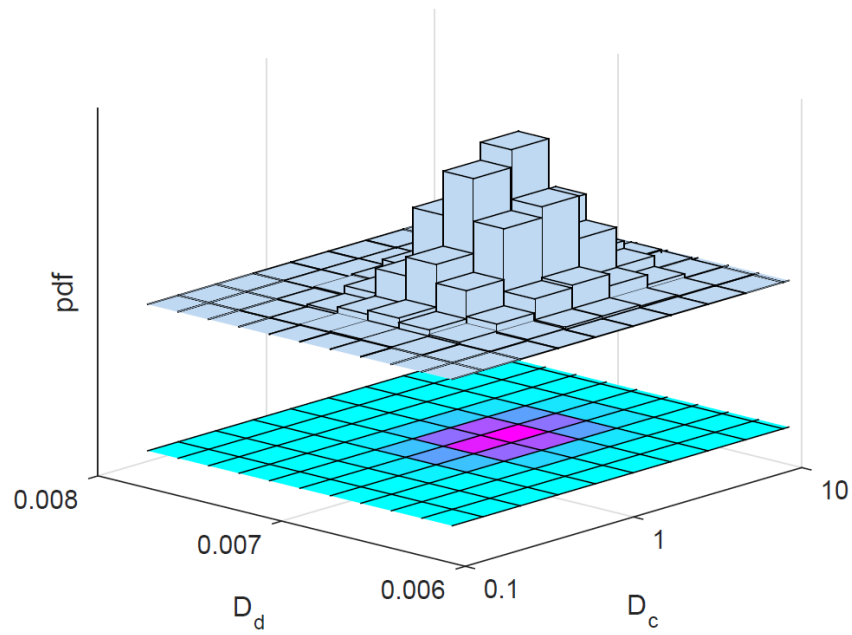


Fig. 67 MCS data histogram and density map for $t=100$ years. No traffic growth.

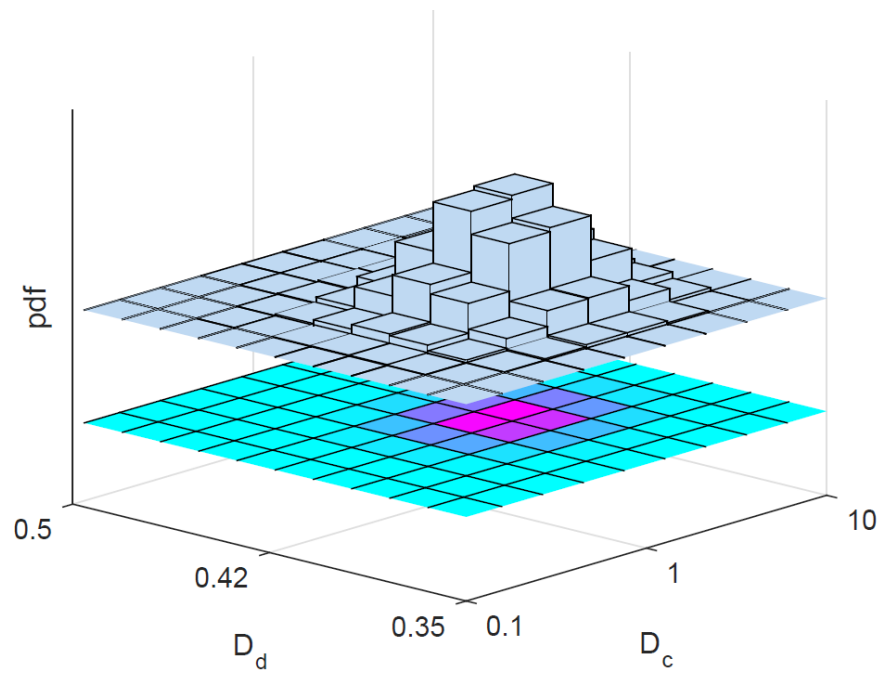


Fig. 68 MCS data histogram and density map for $t=100$ years. Traffic growth scenario 2.

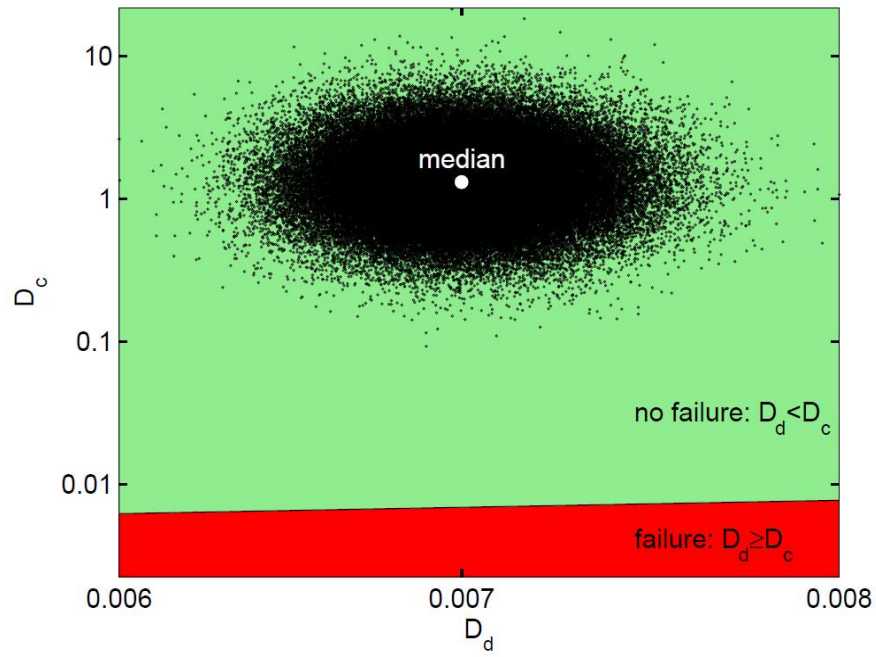


Fig. 69 MCS plot for $t=100$ years. No traffic growth.

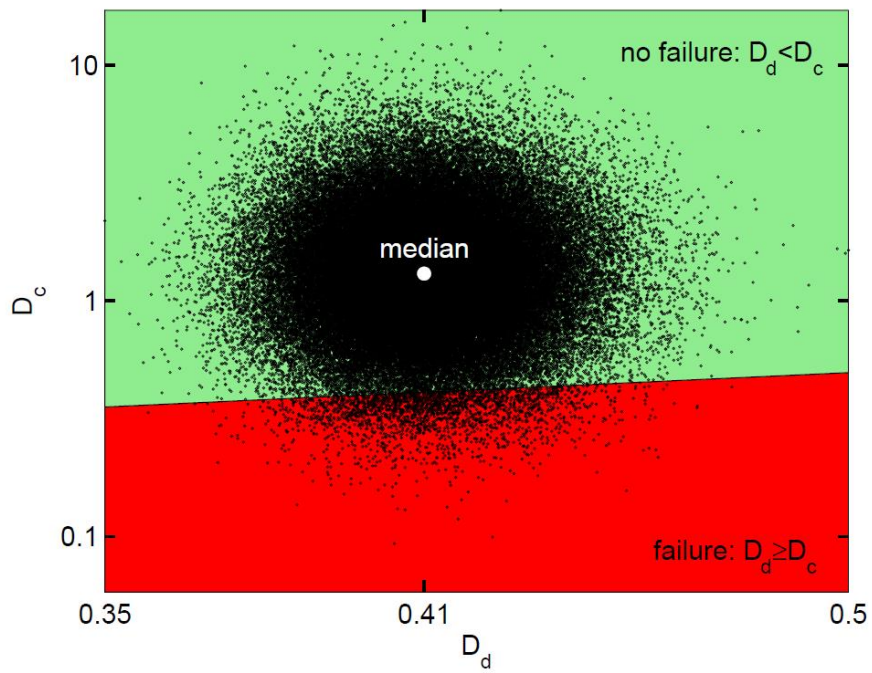


Fig. 70 MCS plot for $t=100$ years. Traffic growth scenario 2.

5.5 Structural evaluation indexes computation

According to SIA 269, the structural evaluation index for type 2, 3, 4 ultimate limit states²⁰ (ULS) is defined as follows:

$$\nu = \frac{R_d}{E_d} \quad (\text{Eq. 65})$$

where:

- R_d is the design value of resistance;
- E_d is the design value of action effect.

In the case of fatigue, different verification schemes are proposed in Eurocode standards, see also section 6.4, the most relevant here being based on damage accumulation :

$$\nu_3 = D_d / D_c \quad (\text{Eq. 66})$$

As an approximation, it could also be expressed in terms of the reliability indexes ratio :

$$\nu_3 \approx \beta_{100} / \beta_{t,100} \quad (\text{Eq. 67})$$

where:

- D_d is the total accumulated damage (at end of design life, or service life; usually 100 years)
- D_c is the critical value of damage sum
- β_{100} is the reliability index after 100 years.
- $\beta_{t,100}$ is the target reliability index after 100 years, taken here as 3.00 (JCSS).

Instead of damage ratio, the index can also be given as the ratio between predicted life and service life. The structural evaluation index, ν_3 was computed for the Venoge bridge study case for the different scenarios. In the case of the actual traffic, scenario 0, the value is very high $\nu_3 \geq 1000 \text{ years} / 100 \text{ years} = 10$ and confirm what was already found, the bridge is very safely designed against fatigue since the actual traffic is low compared to the design one. For scenarios 1 and 2, the values fall below unity, as already seen in Fig. 66 and Tab. 18. For scenario 1 it is approximatively equal to $\nu_3 = 0.89$ and for scenario 2 to $\nu_3 = 0.59$.

5.6 Conclusions and discussion

In this Chapter a new framework for fatigue reliability assessment of existing road bridges was presented, with application to Venoge bridge, a bridge part of the A1 Swiss Motorway.

A two year-continuous registration of WIM data and FE analysis were combined to compute the time-stress response at one assumed bad fatigue critical location (welded cover plate, classified as FAT45 in Eurocode EN1993-1-9 [2]). The probability of failure, P_f , of the fatigue critical detail and related reliability index, β , were computed for a 100 year-design life. The failure probability P_f was computed as the product of $P(E_1|E_2)$ (probability of having critical damage accumulation conditioned on CAFL exceedance)

²⁰ ULS Type 2: Ultimate resistance of structure; ULS Type 3: Ultimate resistance of soil; ULS Type 4: Fatigue.

and $P(E_2)$ (probability of CAFL exceedance). Two different S-N models were used to define fatigue strength of the critical detail: 1) Eurocode S-N model; and 2) ML-MCS S-N model (see Chapter 4).

Assessment of $P(E_1|E_2)$ and $P(E_2)$ asks respectively for the definition of limit state function g_1 and g_2 . Concerning the Eurocode S-N model-based limit state function g_1 (see Equation 53), the random variables m_0 (log-intercept of S-N curve above the CAFL) and D_c (critical damage sum at failure), were defined according to [80] and [43]. The random variable R_d , representing the loading term of the limit state equation, was defined by fitting a Normal distribution R_w (weekly sum of m-power stress ranges) to 104 weekly observations and then applying central limit theorem in order to switch from S_w parameters to S_d parameters. The density plot (see Fig. 58) and the probability plot (see Fig. 59) provide support to the fitted Normal model.

Concerning the ML-MCS S-N model-based limit state function g_1 (see Equation 49), the random S-N model multi-variate vector (including both S-N curve parameters and D_c random variable) was defined according to the results presented in Section 4.4.

Concerning the limit state function g_2 (see Equation 46), the random variable V was defined according to [80], for the Eurocode S-N model-based approach, and according to the results presented in Section 4.4, for the ML-MCS S-N model-based approach. The maximum stress range return level, z_p , representing the loading term of the limit state function, was defined by fitting a GEV family distribution to 104 weekly maximum stress ranges and then computing quantiles of the fitted distribution. Following three diagnostic plots provide support to the fitted GEV model: 1) The return level plot (see Fig. 61); 2) The probability plot (see Fig. 62); and 3) The quantile plot (see Fig. 63).

Failure probability $P(E_1)$ and reliability index β_1 were computed by using FORM with Hasofer-Lind approach, for the Eurocode S-N model-based approach, and by using a MCS scheme, for the ML-MCS S-N model-based approach. Comparison of the Eurocode-based β_1 and the ML-MCS-based β_1 are shown in Tab. 16: for the first 75 years the Eurocode approach is less conservative with respect to ML-MCS approach, while for the last 25 years the Eurocode approach becomes more conservative, both approaches giving high reliability indexes compared to target code values. The difference between the two approaches is due to the combination of three effects: 1) Different values of S-N curve intercept (see Fig. 70); 2) Different position of CAFL (see again Fig. 70); and 3) Approximation of Eurocode approach which uses the characteristic value of CAFL for computing m-power stress ranges (instead of considering CAFL as a random variable).

Failure probability $P(E_2)$ and related reliability index β_2 were computed for both S-N model approaches by using FORM with Hasofer-Lind approach. Comparison of Eurocode S-N model-based β_2 and ML-MCS S-N model-based β_2 are shown in Tab. 17: the Eurocode approach leads to lower values than the ML-MCS approach due to the highly underconservative estimation of CAFL (see Section 3.5). The probability of CAFL exceedance, $P(E_2)$, after 25 years, i.e. onset of damage accumulation, is equal to 56% according to Eurocode approach, while the ML-MCS approach gives $P(E_2)=83\%$, which means that ML-MCS approach indicates the CAFL is most likely to be exceeded after 25 years. For clarification, it is important to recall that ECCS (Europ. Convention for Constructional Steelwork) and later Eurocode EN1993-1-9 S-N curves were established for the design of new structures, not for the assessment of existing ones. Indeed,

simplicity (i.e. having a set of parallel curves) was chosen over more accuracy (different curve slopes, nb of cycles corresponding to CAFL, ...) [40]. The reason for this choice was the efficiency for ranking the details according to their fatigue « reserve » $\Delta\sigma_{Ed}/\Delta\sigma_{C,Rd}$, and thus quickly being able to identify the critical locations. Thus as for new structures the potential unconservatism is « compensated » by proper ranking of criticality of the details (with anyhow large uncertainties on future traffic), it is different for existing structures where remaining fatigue life estimations become predominant. Updated S-N curves shall thus be proposed first for existing bridges.

Fig. 64 shows the evolution of the reliability index β (i.e. the product of both event probabilities) over the 100-year design life : for both Eurocode and ML-MCS S-N model-based approaches, the reliability index is largely greater than β_t EN1990 and β_t JCSS over whole bridge design life.

In order to assess the impact of future traffic evolution on the fatigue analysis, two very aggressive virtual traffic growth scenario were considered. Fig. 66 and Tab. 18 show the evolution of the reliability index β over the 100 year-design life for the two traffic growth scenarios, based on the ML-MCS S-N model. This shows that the year triggering bridge maintenance is shifted by a minimum of 5 to 8 years (it would be more in case of less aggressive scenarios) in function of the target reliability index chosen. These values are only valid for this case study and could result in much larger values in other cases. Discussions in international committees are still going on to fix the most adequate target reliability index or indexes (based on annual failure probability, which value, or at end of service life, depending on consequences, ...).

In conclusion, the reliability analysis framework set up in this Chapter constitutes a powerful tool to perform fatigue reliability analyses of road bridges using WIM-based realistic characterization of traffic loadings. The use of ML-MCS-based S-N model gives more realistic characterization of the fatigue strength with respect to the standard approach and allows to improve the confidence in fatigue reliability analysis. The considered Venoge bridge study case uses 2006-2007 WIM data and is based on stationary load sequences ; however any additional WIM data-set can be used to update estimated model and to take in account possible future traffic growth (non-stationary loading sequences) : in this case probability distributions having time-dependent parameters have to be used for reliability analysis.

6 Calibration of fatigue partial safety factors

6.1 Introduction

In this chapter, a new framework for re-calibration of fatigue checks partial safety factors in standards is proposed. It is built on the results from the previous chapters and allows to improve the confidence in fatigue assessment of existing bridges, as well as design of new ones. It shows to code developers how one could revise the standards, such as Eurocodes, for the different fatigue check schemes and associated partial safety factors. Again, the framework developed has general applicability, but after presenting it in a generic way, it will herein be applied to two study cases of welded joints for recalibration of the partial resistance factor values of the Eurocode EN 1993-1-9.

The safety and serviceability issues in structural design are addressed in structural codes by defining design equations which compare load effects and resistance. Due to the uncertain nature of load and resistance terms, these are modeled as random variables. Characteristic values of random variables and partial safety factors are introduced in order to achieve a target reliability level. The process of choosing design format, characteristic values, partial safety factors and target reliability level, by using some reliability methods, is called code calibration. The problem of reliability based code calibration has been addressed by many authors, [25],[66],[27] and it has also been implemented in Eurocode EN1990 [1].

The typical design equation for the verification of a structural component (see Eurocode EN1990) is:

$$G = \frac{zR_c}{\gamma_M} - (\gamma_{F1}E_{c1} + \dots + \gamma_{Fn}E_{cn}) = 0 \quad (\text{Eq. 68})$$

where:

- R_c is the characteristic value of resistance;
- z is the design factor;
- γ_M is the partial safety factor for resistance;
- E_{ci} is the characteristic value of the i^{th} action effect;
- γ_{Fi} is the partial safety factor for the i^{th} action effect.

The application of partial safety factors together with characteristic values of resistance and loading effects ensures a certain level of reliability for the designed structural component.

According to the design Equation 68 a reliability analysis can be made with the following limit state equation:

$$g = zR - (E_1 + \dots + E_n) = 0 \quad (\text{Eq. 69})$$

For a given probabilistic model of the basis random variables R and E_i , and for an assigned value of target failure probability, the optimal design $\hat{z}$ can be computed, which corresponds to the required level of target reliability. Having determined the optimal design $\hat{z}$, the corresponding design point (R_d, E_{di}) can be computed and the partial safety factors can be derived as follows:

$$\gamma_M = \frac{R_c}{R_d} \quad (\text{Eq. 70})$$

$$\gamma_{Fi} = \frac{E_{di}}{E_{ci}} \quad (\text{Eq. 71})$$

The relation between design values, characteristic values and partial safety factors is illustrated in Fig. 71, for the simple case of only one action effect.

Resolution of the code calibration problem asks for: 1) Determination of target reliability level; and 2) Calibration of partial safety factors.

The choice of target reliability level is generally based on an economic decision theory approach. Discussions in international committees are still going on to fix the most adequate annual target reliability index, depending on economic and failure consequences. The target reliability index determination issue is not addressed in this study: annual target reliability index recommended in JCSS PMC [43] is used.

The calibration of partial safety factors is a decision problem, in which partial safety factors are decision variables which are calibrated by maximizing an objective function. Faber et al. [27] proposed a practical approach for calibration of partial safety factors, in which the objective function is formulated as follows:

$$W(\gamma) = \sum_{j=1}^L w_j \cdot (\beta_j(\gamma) - \beta_t) \quad (\text{Eq. 72})$$

where:

- L is the number of load cases;
- w_j are the importance factors of different design load cases;
- β_t is the target reliability index.

Partial safety factors γ are computed by minimizing the objective function in Equation 72, in which the reliability index is computed by solving the limit state equation (see Equation 69), having determined the optimal design factor $\hat{z}$ from the design equation (see Equation 68).

It is noted that when partial safety factors are calibrated from Equation 72 they are not independent and in the case with one resistance factor and one loading factor only the product of them can be calculated. General limit state Equation 69 and general design Equation 68 can be easily adapted to the fatigue design case.

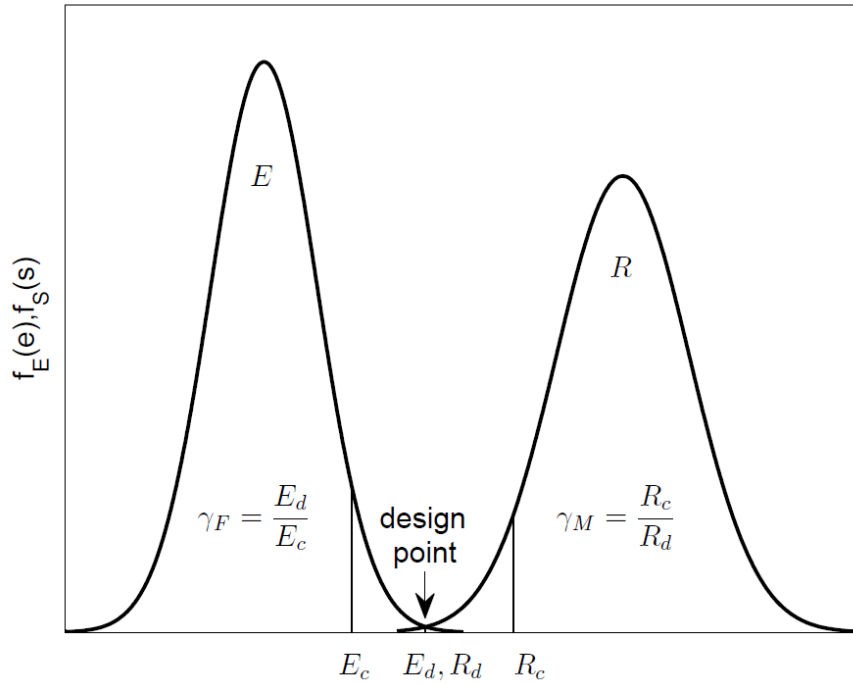


Fig. 71 Relation between characteristic values, design values and partial safety factors.

In Eurocode standards, three verification schemes are proposed for fatigue design under VA loadings (see A.6 of Eurocode EN-1993-1-9 [2]):

1. Verification scheme based on CAFL;
2. Verification scheme based on CA equivalent stress range at $2 \cdot 10^6$ cycles;
3. Verification scheme based on accumulated damage.

Verification scheme 1: based on CAFL

The fatigue design based on CAFL exceedance has to meet the following criterion:

$$\gamma_{Ff} \cdot S_{max_c} \leq \frac{CAFL}{\gamma_{Mf}} \quad (\text{Eq. 73})$$

where:

- CAFL is the characteristic value of the fatigue limit;
- S_{max_c} is the characteristic value of maximum loading stress range;
- γ_{Ff} is the loading partial safety factor, which is set to 1.0;
- γ_{Mf} is the resistance partial safety factor.

Verification scheme 2: based on CA equivalent stress range

The fatigue design based on CA equivalent stress range has to meet the following criterion:

$$\gamma_{Ff} \cdot S_{E,2} \leq \frac{S_c}{\gamma_{Mf}} \quad (\text{Eq. 74})$$

where:

- $S_{E,2}$ is the equivalent stress range, reported at $2 \cdot 10^6$ cycles, which is computed by using Fatigue Load Model (FLM) 3 and λ damage equivalent factors;
- S_c is the characteristic fatigue strength at $2 \cdot 10^6$ cycles (FAT).

Verification scheme 3: based on damage accumulation

The fatigue design based on damage accumulation has to meet the following criterion:

$$D_d = \sum_i^{N_{tot}} \frac{n_i}{N_i} \leq 1.0 \quad (\text{Eq. 75})$$

where:

- n_i is the number of cycles corresponding to the design loading stress range $\gamma_{Ff} \cdot S_i$;
- N_i is the number of cycles to failure obtained from the factored $\frac{S_c}{\gamma_{Mf}} - N$ curve.

The definition of the partial resistance factor γ_{Mf} in EN 1993-1-9 (Section 1.4, pp. 9, [2]) is ambiguous because γ_{Mf} is strictly defined for fatigue strength at $2 \cdot 10^6$ cycles (therefore relevant only for verification scheme 2), but it is generically applied for fatigue strengths at any number of cycles (verification schemes 1 and 3).

Recommended values of partial factor γ_{Mf} are presented in Tab. 19. Proof of calibration of these values is not available and the real reliability level, β , corresponding to the values recommended in Tab. 19 is still under debate.

Tab. 19 Recommended values for γ_{Mf} , by assuming $\gamma_{Ff} = 1.0$ (Table 3.1 of [2])

Design method	Consequence of failure	
	Low consequence	High consequence
Damage tolerant	1.00	1.15
Safe life	1.15	1.35

In this Chapter a new framework for calibration of fatigue partial safety factors is presented.

It is built on the results from the previous chapters and allows to improve the confidence in fatigue design of new bridges, as well as assessment of existing ones. It shows to code developers how one could revise the standards, such as Eurocodes, for fatigue checks of structures and associated partial safety factors. Again, the framework developed has

general applicability, but after presenting it in a generic way, it will herein be applied to two study cases of welded joints for recalibration of the partial resistance factor values of the Eurocode EN 1993-1-9. The Chapter is organized as follows:

- in Section 6.2 the new framework for calibration of fatigue partial safety factors using ML-MCS S-N model is presented;
- in section 6.3 an application of the framework to two typical bridge fatigue sensitive welded joints is considered; Results of analyses of the two considered study cases are presented. A practical example, showing the impact of the choice of design S-N curves on fatigue design of a road bridge, is also given;
- in Section 6.3.2 the structural evaluation index for fatigue damage accumulation, ν_3 is computed for the Venoge bridge study case;
- in Section 6.5 results are discussed and comparison with partial safety factor values recommended in Eurocode EN1993-1-9 is made.

6.2 New framework for partial safety factor calibration

The framework uses as previously a ML-MCS S-N model. It includes the three verification schemes which have been presented in Section 6.1. One different limit state function is formulated for each of verification schemes.

Partial safety factors are calibrated for the three different design verification schemes, by using following general objective function (which is valid for all three verifications):

$$W(\gamma_{Ff}, \gamma_{Mf}) = \sum_{j=1}^L w_j \cdot (\beta_j(\gamma_{Ff}, \gamma_{Mf}) - \beta_t)^2 \quad (\text{Eq. 76})$$

where:

- γ_{Ff} is the partial safety factor for fatigue loading;
- γ_{Mf} is the partial safety factor for fatigue resistance;
- L is the number of load cases;
- w_j are the load case importance factors;
- β_j are the computed reliability indexes, for $j = 1, \dots, L$;
- $\beta_t = -\Phi^{-1}([\Phi(4.2)]^{100}) = 3.00$, is the target reliability index for a 100 year reference period²¹;

²¹ Based on $\beta_t^{1y} = 4.2$ [43], assuming independency.

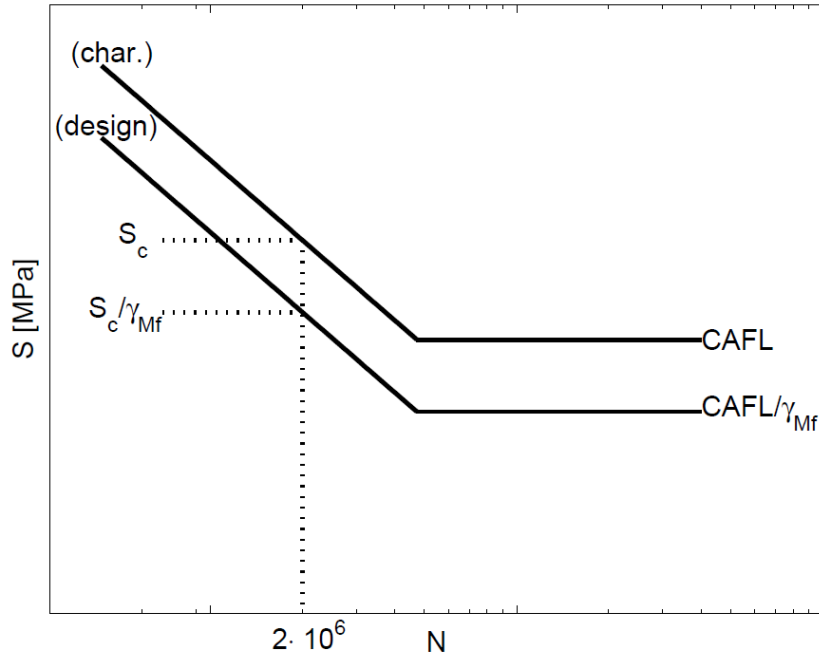


Fig. 72 Resistance partial safety factor, verification schemes 1 and 2.

The reliability indexes β_j are computed by solving the limit state equation for different load cases ($j=1, \dots, L$), after having determined the optimal design, $\hat{z}$, as solution of the design equation. Formulation of limit state and design equations for the three considered verification cases is discussed in following three subsections.

6.2.1 Verification scheme 1: based on CAFL

The design equation, for the generic load case j , is formulated as follows:

$$G_j = z_j \cdot \frac{\text{CAFL}}{\gamma_{Mf}} - \gamma_{Ff} \cdot S_{\max_{c,j}} \quad (\text{Eq. 77})$$

where z_j is the design factor and $S_{\max_{c,j}}$ is the characteristic value of the maximum stress range, $S_{\max,j}$. The geometrical interpretation of the resistance partial safety factor γ_{Mf} is illustrated in Fig. 72. $S_{\max}$ is modeled as a Gumbel random variable, $\mathcal{G}_1(\mu_{\mathcal{G}_1}, \sigma_{\mathcal{G}_1})$, having coefficient of variation equal to:

$$c_{v\mathcal{G}_1} = \frac{1.28 (\sigma_{\mathcal{G}_1} / \mu_{\mathcal{G}_1})}{0.58 (\sigma_{\mathcal{G}_1} / \mu_{\mathcal{G}_1}) + 1} \quad (\text{Eq. 78})$$

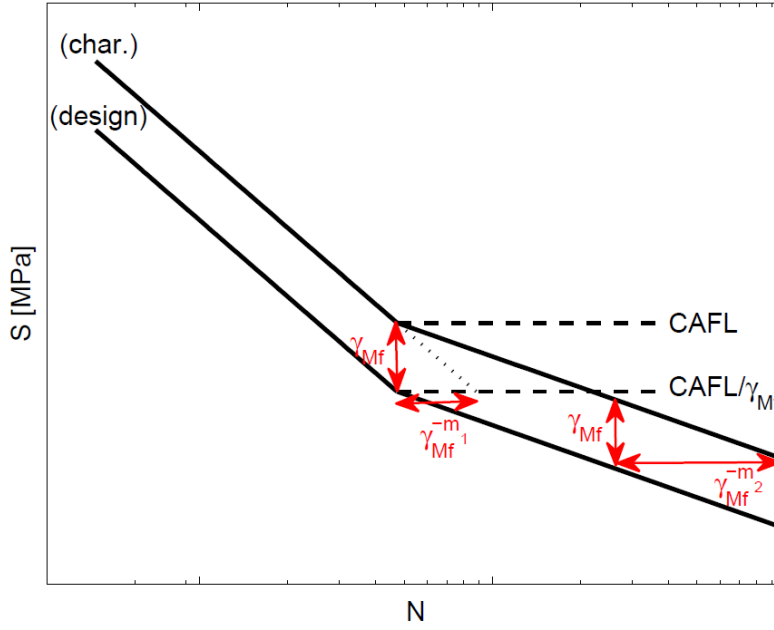


Fig. 73 Resistance partial safety factor, verification scheme 3.

In order to take into account both the effects of position and dispersion of loading term, 20 load cases are built by considering all possible combinations of the following values of μ_{G_1} and c_{vG_1} :

$$\mu_{G_1} = (\exp(v_{.05}), \dots, \Delta, \dots, \exp(\mu_V)) \quad \text{with } \Delta = (\exp(\mu_V) - \exp(v_{.05})) / 4 \quad (\text{Eq. 79})$$

$$c_{vG_1} = (0.20, 0.27, 0.33, 0.40) \quad (\text{Eq. 80})$$

The optimal design, $\hat{z}_j$, for the load case j , is computed by solving the design Equation 77, $G_j = 0$, for assigned values of partial safety factors.

The reliability index β_j is then computed by solving the following limit state equation²²:

$$g_j = \hat{z}_j \cdot \exp(V) - S_{max,j} \quad (\text{Eq. 81})$$

Fatigue partial safety factors are calibrated by minimizing following objective function:

$$W(\gamma) = \sum_{j=1}^{20} w_j \cdot (\beta_j(\gamma) - 3.00)^2 \quad (\text{Eq. 82})$$

where $w_j = 1$ for $j = 1, \dots, 20$, since all load cases have the same importance.

It is noted that when γ_{Ff} and γ_{Mf} are determined with Equations 77, 81 and 82, they are not independent and only the product $\gamma_{Ff} \cdot \gamma_{Mf}$ is determined.

²² It is noted that both the epistemic uncertainty of the model parameters and the aleatory randomness of the CAFL are taken into account in the definition of the fatigue limit state function.

6.2.2 Verification scheme 2: based on CA equivalent stress range

The design equation, for the generic load case j , is formulated as follows:

$$G_j = z_j \cdot \frac{S_c}{\gamma_{Mf}} - \gamma_{Ff} \cdot S_{E,2c,j} \quad (\text{Eq. 83})$$

where z_j is the design factor, S_c is the characteristic value of fatigue strength at $2 \cdot 10^6$ cycles, and $S_{E,2c,j}$ is the characteristic value of equivalent stress range at $2 \cdot 10^6$ cycles, $S_{E,2,j}$.

The geometrical interpretation of the resistance partial safety factor is illustrated in Fig. 72.

$S_{E,2}$ is modeled as a log-Normal random variable²³, $\log \mathcal{N}_2(\mu_{\mathcal{N}_2}, \sigma_{\mathcal{N}_2})$, having coefficient of variation equal to:

$$c_{v_{\mathcal{N}_2}} = \sqrt{\exp(\sigma_{\mathcal{N}_2}^2) - 1} \quad (\text{Eq. 84})$$

In order to take into account both the effects of position and dispersion of loading term, 20 load cases are built by considering all possible combinations of following values of $\mu_{\mathcal{N}_2}$ and $c_{v_{\mathcal{N}_2}}$:

$$\mu_{\mathcal{N}_2} = (3.40, 3.57, 3.75, 3.92, 4.10) \quad (\text{Eq. 85})$$

$$c_{v_{\mathcal{N}_2}} = (0.10, 0.13, 0.17, 0.20) \quad (\text{Eq. 86})$$

The optimal design, $\hat{z}_j$, for the load case j , is computed by solving the design Equation 83, $G_j = 0$, for assigned values of partial safety factors.

²³ Modeling the logarithm of the equivalent stress range as a Normal random variable allows for consideration of the variability on the maximum stress range obtained from fatigue load model as well as of the variabilities on different λ factors.

The reliability index β_j is then computed by solving the following limit state equation²⁴:

$$g_j = \hat{z}_j \cdot S - S_{E,2,j} \quad (\text{Eq. 87})$$

Fatigue partial safety factors are calibrated by minimizing the following objective function:

$$W(\gamma) = \sum_{j=1}^{20} w_j \cdot (\beta_j(\gamma) - 3.00)^2 \quad (\text{Eq. 88})$$

where $w_j = 1$ for $j = 1, \dots, 20$, since all load cases have the same importance.

It is noted again that when γ_{Ff} and γ_{Mf} are determined with Equations 83, 87 and 88, they are not independent and only the product $\gamma_{Ff} \cdot \gamma_{Mf}$ is determined.

6.2.3 Verification scheme 3: based on damage accumulation

The design equation, for the generic load case j , is formulated as follows:

$$G_j = d_{c,5} - \left\{ \int_{\frac{CAFL}{\gamma_{Mf}}}^{\infty} \frac{n_{totc,j} \cdot \frac{s \cdot \gamma_{Ff}}{2\lambda_R^2} \exp\left(-\frac{(s \cdot \gamma_{Ff})^2}{2\lambda_R^2}\right)}{\frac{z_j \cdot N_c(s)}{(\gamma_{Mf})^{-m_1}}} ds + \int_0^{\frac{CAFL}{\gamma_{Mf}}} \frac{n_{totc,j} \cdot \frac{s \cdot \gamma_{Ff}}{2\lambda_R^2} \exp\left(-\frac{(s \cdot \gamma_{Ff})^2}{2\lambda_R^2}\right)}{\frac{z_j \cdot N_c(s)}{(\gamma_{Mf})^{-m_2}}} ds \right\} \quad (\text{Eq. 89})$$

where z_j is the design factor, $d_{c,5}$ is the characteristic value of the critical damage, $n_{totc,j}$ is the characteristic value of the total number of cycles and λ_R is the scale parameter of the Rayleigh loading spectrum.

The geometrical interpretation of the resistance partial safety factor is illustrated in Fig. 73.

n_{tot} is modeled as a Gumbel random variable, $\mathcal{G}_3(10^8, \sigma_{\mathcal{G}_3})$, having a coefficient of variation $c_{v\mathcal{G}_3}$.

In order to take into account both the effects of uncertainty on loading spectrum stress ranges and uncertainty on total number of cycles, and since more simulations are needed to get convergence 32 load cases²⁵ (instead of 20) are built by considering all possible combinations of the following values of λ_R and $c_{v\mathcal{G}_3}$:

$$\lambda_R : \quad \varsigma = (0.05, 0.08, 0.11, \dots, 0.25) \quad (\text{Eq. 90})$$

$$c_{v\mathcal{G}_3} = (0.20, 0.27, 0.33, 0.40) \quad (\text{Eq. 91})$$

²⁴ It is noted that both the epistemic uncertainty of the model parameters and the aleatory randomness of the fatigue life are taken into account in the definition of the fatigue limit state function.

²⁵ This was found to be numerically more efficient than keeping 20 load cases and increase the number of simulations per load case in solving the MCS scheme. It does not affect the final result.

where ζ is the percentage of stress range cycles exceeding the characteristic value of the CAFL (see Fig. 74). Rayleigh distribution was chosen to model the loading spectrum since this probability distribution provides a quite accurate characterization of the upper tail of the spectrum, with only one distribution parameter.

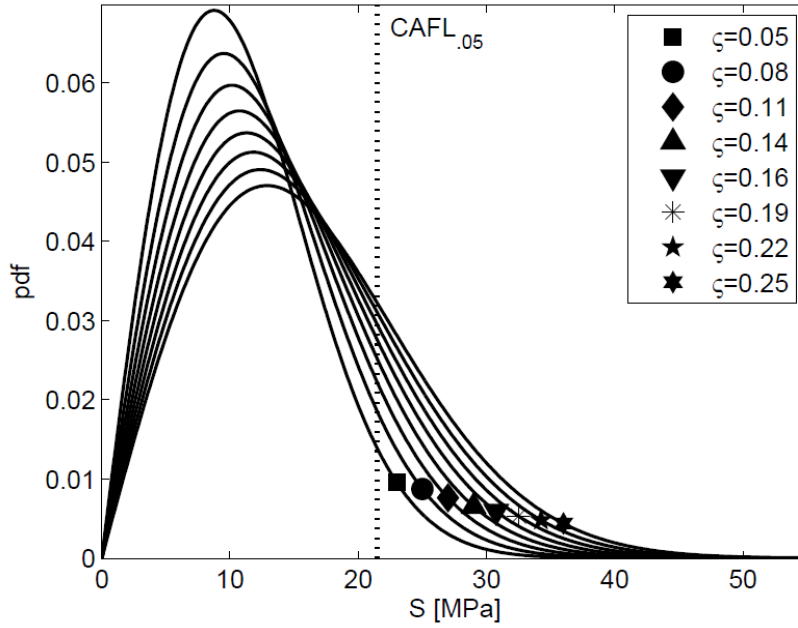


Fig. 74 Rayleigh loading spectra, cover plate.

The optimal design, $\hat{z}_j$, for the load case j , is computed by solving the design Equation 89, $G_j = 0$, for assigned values of partial safety factors. The reliability index β_j is then computed by solving the following limit state equation²⁶:

$$g_j = D_c - \left\{ \int_{\exp(V)}^{\infty} \frac{n_{totj} \cdot \frac{s}{\lambda_R^2} \exp\left(-\frac{s^2}{2\lambda_R^2}\right)}{\hat{z}_j \cdot N(s)} ds + \int_0^{\exp(V)} \frac{n_{totj} \cdot \frac{s}{\lambda_R^2} \exp\left(-\frac{s^2}{2\lambda_R^2}\right)}{\hat{z}_j \cdot N_c(s)} ds \right\} \quad (\text{Eq. 92})$$

Fatigue partial safety factors are calibrated by minimizing the following objective function:

$$W(\gamma) = \sum_{j=1}^{32} w_j \cdot (\beta_j(\gamma) - 3.00)^2 \quad (\text{Eq. 93})$$

where $w_j = 1$ for $j = 1, \dots, 32$, since all load cases have the same importance.

As for the previous two verification schemes, it is noted here that when γ_{Ff} and γ_{Mf} are determined with Equations 89, 92 and 93, they are not independent and only a combination of them is determined.

²⁶ It is noted that both the epistemic uncertainty of the model parameters and the aleatory randomness of the CAFL, of the fatigue life and of the critical damage sum, are taken into account in the definition of the fatigue limit state function.

The framework presented in this Section was implemented in the Matlab Toolbox TB3 (see Appendix III). The Matlab Toolbox TB3 was validated using the structural reliability analysis software STRUREL [67].

6.3 Example of calibration and comparisons

6.3.1 Study cases

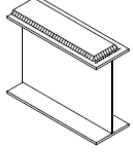
The framework built in Section 6.2 was applied to the two fatigue sensitive details already studied in Chapters 3 and 4: 1) Welded cover plate; and 2) Welded in-plane gusset. The VA S-N stochastic model of the two considered details is given in Section 4.4.

The parameters of VA S-N curves are resumed in Tab. 20 for the convenience of the reader. It is recalled that the cover plate detail and the in-plane gusset detail were classified respectively as FAT45 and FAT50 in Eurocode EN1993-1-9 [2].

Verification scheme 1: based on CAFL

In Section 6.2.1 the procedure for calibrating fatigue partial safety factors for verification scheme 1 was presented. It was noted that only the product of $\gamma_{Ff} \cdot \gamma_{Mf}$ could be computed, since the resistance and load partial safety factors were not independent. Moreover it is possible to set the load partial safety factor equal to 1.0 by proper choice of the return period of the characteristic value of the maximum stress range, $\rho_{S_{\max}}$.

Tab. 20 VA S-N model parameters

		
$Y V$	$\mathcal{N}\left(\mu_{Y V}, \exp(\sigma)\right)$	$\mathcal{N}\left(\mu_{Y V}, \exp(\sigma)\right)$
V	$\mathcal{N}\left(\mu_V, \exp(\sigma_V)\right)$	$\mathcal{N}\left(\mu_V, \exp(\sigma_V)\right)$
m_0	$\mathcal{N}(28.66, 2.85)$	$\mathcal{N}(25.77, 0.95)$
m_1	$\mathcal{N}(-3.42, 0.79)$	$\mathcal{N}(-2.67, 0.21)$
σ	$\mathcal{N}(-0.54, 0.20)$	$\mathcal{N}(-1.05, 0.14)$
μ_V	$\mathcal{N}(3.45, 0.06)$	$\mathcal{N}(3.86, 0.13)$
σ_V	$\mathcal{N}(-1.62, 0.41)$	$\mathcal{N}(-1.67, 0.50)$
Δm	6	5
D_c	$\log \mathcal{N}(0.27, 0.58)$	$\log \mathcal{N}(-0.31, 0.60)$
$d_{c,0.5}$	1.31	0.74
CAFL	21MPa	30MPa
a_{05}	27.58	25.14

The resistance partial safety factor $\hat{\gamma}_{Mf}$ and the return period $\hat{\rho}_{S_{\max}}$, corresponding to $\gamma_{Ff} = 1.0$, were computed using the following scheme:

1. The resistance partial safety factor $\hat{\gamma}_{Mf} = \gamma_{Mf}(\gamma_{Ff} = 1.0)$ was computed by minimizing the objective Equation 82, where the reliability indexes β_j were computed by setting $S_{\max, j} = S_{\max_{c, j}}$ in Equation 81;
2. The γ – characteristic return period plot (see Fig. 75), which relates the product $\gamma_{Mf} \cdot \gamma_{Ff}$ to the return period of the characteristic value of the maximum stress range, $\rho_{S_{\max}}$, was produced by minimizing the objective Equation 82, where the reliability indexes β_j were computed by considering $\rho_{S_{\max}} = (250, 500, \dots, 2000)$ in Equation 81;
3. The return period $\hat{\rho}_{S_{\max}}$ was computed by intersecting the γ – characteristic return period plot with the straight line $\gamma_{Ff} = 1.0$ (see again Fig. 75).

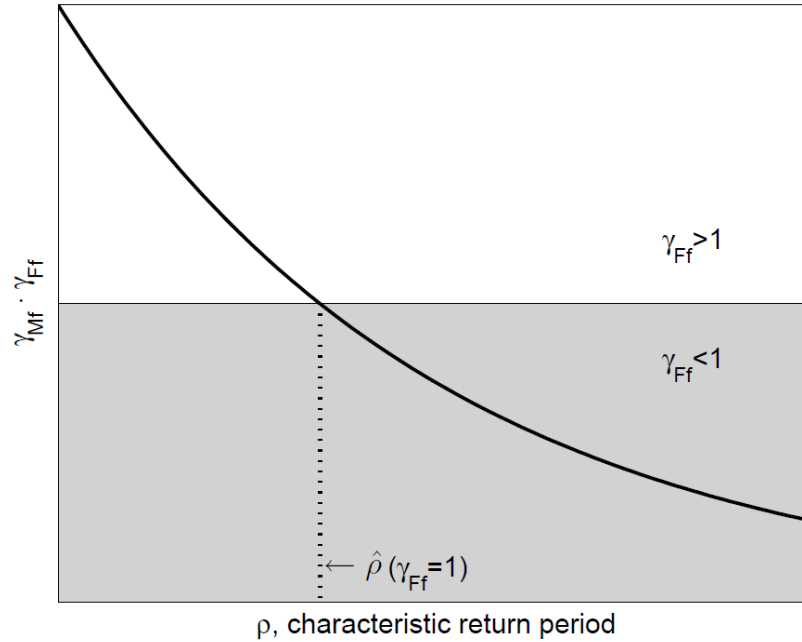


Fig. 75 Illustration of γ – characteristic return period plot.

Verification scheme 2: based on CA equivalent stress range

In Section 6.2.2 the procedure for calibrating fatigue partial safety factors for verification scheme 2 was presented. Also for this verification scheme, it was noted that only the product of $\gamma_{Mf} \cdot \gamma_{Ff}$ could be computed, since the resistance and loading safety factors were not independent.

Moreover, as for the verification scheme 1, a three step-scheme was set in order to compute the resistance partial safety factor $\hat{\gamma}_{Mf}$ and the return period $\hat{\rho}_{S_{E,2}}$, corresponding to $\gamma_{Ff} = 1.0$. The scheme is the same as the one used for the verification scheme 1, except for objective Equation 88 and limit state Equation 87 which were used in place of respectively objective Equation 82 and limit state Equation 81.

Verification scheme 3: based on damage accumulation

In Section 6.2.3 the procedure for calibrating fatigue partial safety factors for verification scheme 3 was presented. Also for this verification scheme, it was noted that only a combination of $\gamma_{Mf} \cdot \gamma_{Ef}$ could be computed, since the resistance and loading safety factors were not independent.

Moreover, as for the verification schemes 1 and 2, a three step-scheme was set in order to compute the resistance partial safety factor $\hat{\gamma}_{Mf}$ and the return period $\hat{\rho}_{S_{Ntot}}$, corresponding to $\gamma_{Ef} = 1.0$. The scheme is the same as the one used for the verification scheme 1, except for objective Equation 93 and limit state Equation 92 which were used in place of respectively objective Equation 82 and limit state Equation 81.

6.3.2 Results

The resistance partial safety factor, $\hat{\gamma}_{Mf}$, and the characteristic return period, $\hat{\rho}$, which allow to set $\gamma_{Ef} = 1.0$, are shown in Tab. 21, for the three verification schemes.

An example of the graphical determination of the characteristic return period is given in Fig. 76, for the cover plate, verification scheme 1. For each verification scheme, the fatigue verification criterion is also presented in Tab. 21. It has to be noted that for the verification scheme 3 (damage accumulation), the computed resistance partial safety factors refer to the characteristic VA S-N curve determined at $D_c = d_{c,5} \neq 1.0$, and the critical damage used in the fatigue verification criterion is $d_{c,5} \neq 1.0$. For this reason results for verification scheme 3 can not be directly compared to Eurocode standards, in which $d_{c,5} = 1.0$.

In Tab. 22, results for verification schemes 1 and 2 are compared to Eurocode standards in terms of design value of CAFL and of design value of fatigue strength at $2 \cdot 10^6$ cycles. Since crack formation in the considered details could rapidly lead to structural failure, the case *Safe life, High consequences* is chosen in Table 3.1 of EN 1993-1-9 ($\gamma_{Mf} = 1.35$).

Results for verification scheme 3 are compared to Eurocode standards in terms of design VA S-N curves, in Figures Fig. 77 and Fig. 78. In order to make direct comparison with Eurocode design S-N curves, the ML-MCS design S-N curves were re-scaled at $d_{c,5} = 1.0$ (see Equation 38).

The calibration shows, for both details :

- Partial safety factors for verification scheme 1 are found higher than for the scheme 2, and higher than the Eurocode EN1993-1-9 values ;
- Partial safety factors values for verifications scheme 2 are similar to Eurocode EN1993-1-9 safest case, i.e. $\gamma_{Mf} = 1.35$.

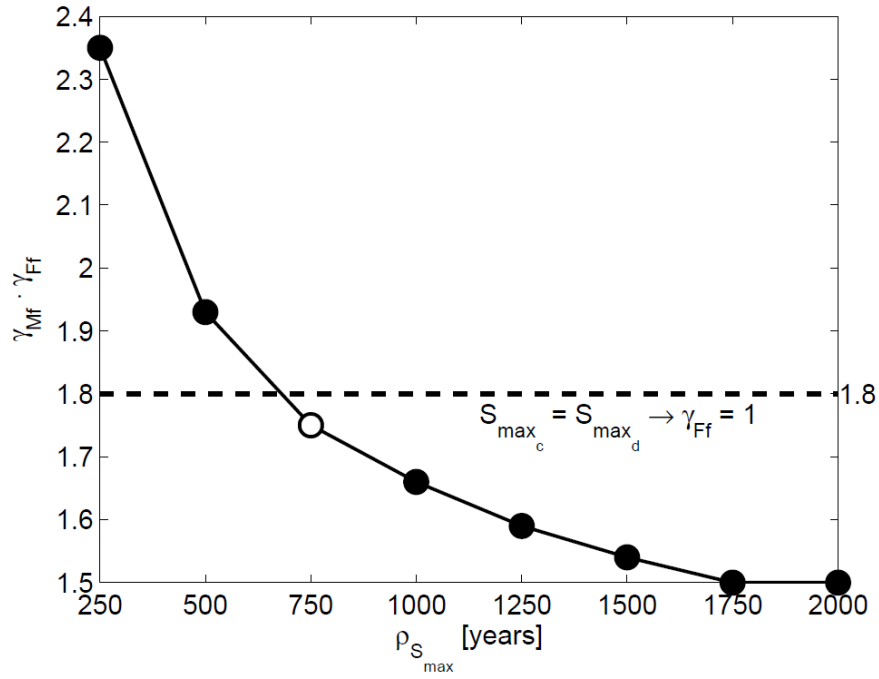
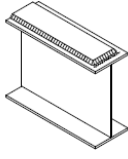
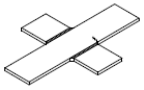
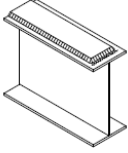
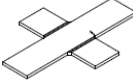


Fig. 76 Return level plot, cover plate, verification scheme 1.

Tab. 21 Resistance partial safety factors and related characteristic return periods ($\gamma_{Mf} = 1.0$)

		
1	$S_{max} \leq \frac{CAFL}{\gamma_{Mf}}$ $\hat{\gamma}_{Mf} = 1.80$ $\hat{\rho}_{S_{max}} = 750 \text{ years}$	$S_{max} \leq \frac{CAFL}{\gamma_{Mf}}$ $\hat{\gamma}_{Mf} = 2.27$ $\hat{\rho}_{S_{max}} = 500 \text{ years}$
2	$S_{E,2} \leq \frac{S_c}{\gamma_{Mf}}$ $\hat{\gamma}_{Mf} = 1.33$ $\hat{\rho}_{S_{E,2}} = 750 \text{ years}$	$S_{E,2} \leq \frac{S_c}{\gamma_{Mf}}$ $\hat{\gamma}_{Mf} = 1.26$ $\hat{\rho}_{S_{E,2}} = 1000 \text{ years}$
3	$\sum_i^{n_{tot}} \frac{n_i}{N(\gamma_{Mf})} \leq 1.31$ $\hat{\gamma}_{Mf} = 1.37$ $\hat{\rho}_{n_{tot}} = 500 \text{ years}$	$\sum_i^{n_{tot}} \frac{n_i}{N(\gamma_{Mf})} \leq 0.74$ $\hat{\gamma}_{Mf} = 1.45$ $\hat{\rho}_{n_{tot}} = 500 \text{ years}$

Tab. 22 Comparison of design values for verification schemes 1 and 2

					
		ML-MCS	Eurocode	ML-MCS	Eurocode
1	CAFL [MPa]	21	33	31	29
	CAFL _d [MPa]	12	24	14	21
2	S _c [MPa]	46	45	53	40
	S _{c,d} [MPa]	35	33	42	30

6.3.3 Comparison of fatigue assessments based on verification scheme 3

In this section a comparison of fatigue assessment of a road bridge using damage accumulation criterion (verification scheme 3) is given. This example allows to evaluate the impact of the choice of design S-N curves (i.e. characteristic curves and partial resistance factors) on the computed fatigue life.

The Venoge bridge, which was studied in Section 5.3, is considered again in this comparison example.

Two critical details are considered:

1. Welded cover-plate, on the lower flange of the enlargement left bridge at the first mid-span;
2. In-plane welded gusset, on the lower flange of the enlargement left bridge at the first mid-span. This detail does not exist in reality and is considered for academic purpose only.

In order to define the stress range loading spectrum, a Rayleigh probability distribution is fitted to observed stress ranges at the critical section in the year 2006. The Rayleigh stress range spectrum, having scale parameter equal to 5.3, is shown in Fig. 79 (a).

The design value of allowed number of cycles, n_d , is computed, for the two considered critical details, using following equation:

$$n_d = \frac{d_{c,.5}}{\int_0^\infty \frac{f_S(s)}{N(s)} ds} \quad (\text{Eq. 94})$$

where:

- $d_{c,.5}$ is the characteristic value of critical damage sum. When ML-MCS approach is used, $d_{c,.5}$ is equal to 1.31 for the cover plate detail and it is equal to 0.74 for the in-plane gusset detail; when Eurocode approach is used $d_{c,.5}$ is equal to 1.0 for both critical details;
- $f_S(s)$ is the Rayleigh loading stress range spectrum;
- $N(s)$ is the endurance obtained from the factored $\frac{S_c}{\gamma_{Mf}} - N$ curve. When ML-MCS approach is used γ_{Mf} is equal to 1.37 for the cover plate details and it is equal to 1.45

for the in-plane gusset detail (see Tab. 21); when Eurocode approach is used γ_{Mf} is equal to 1.35 for both critical details.

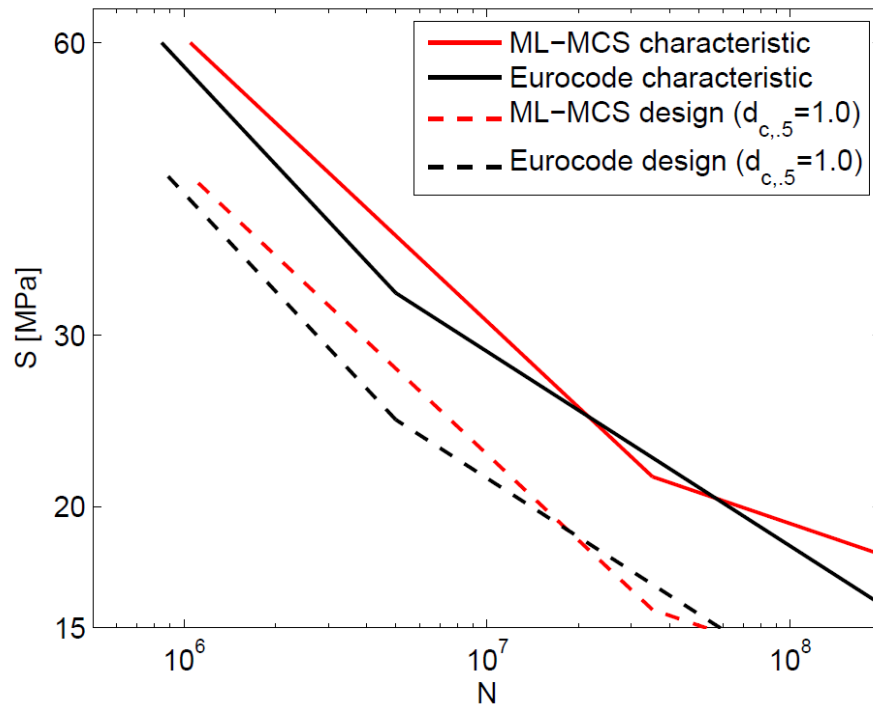


Fig. 77 Comparison of VA design S-N curves for verification scheme 3, cover plate.

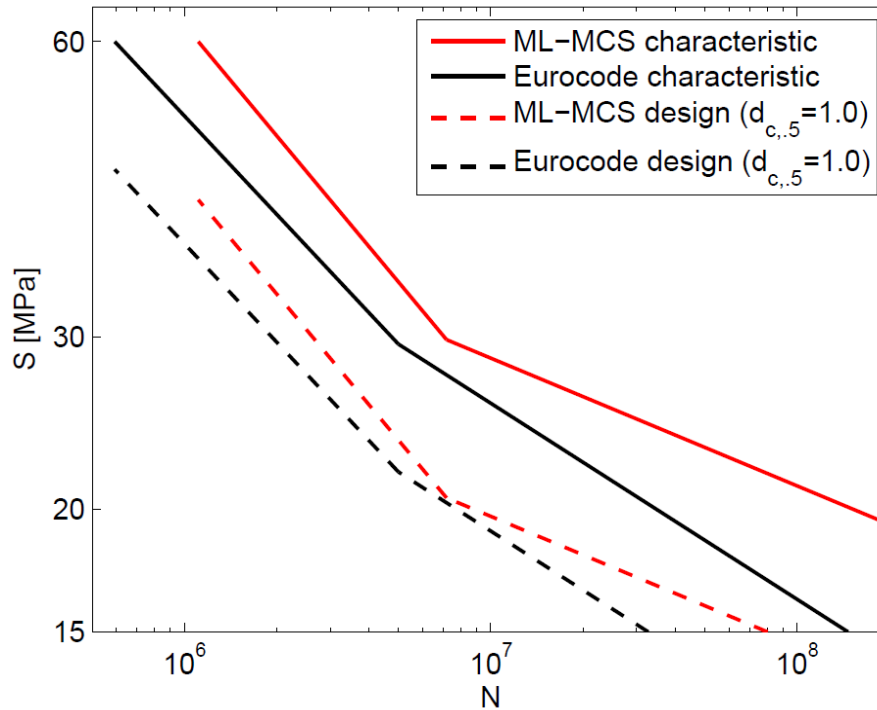


Fig. 78 Comparison of VA design S-N curves for verification scheme 3, in-plane gusset.

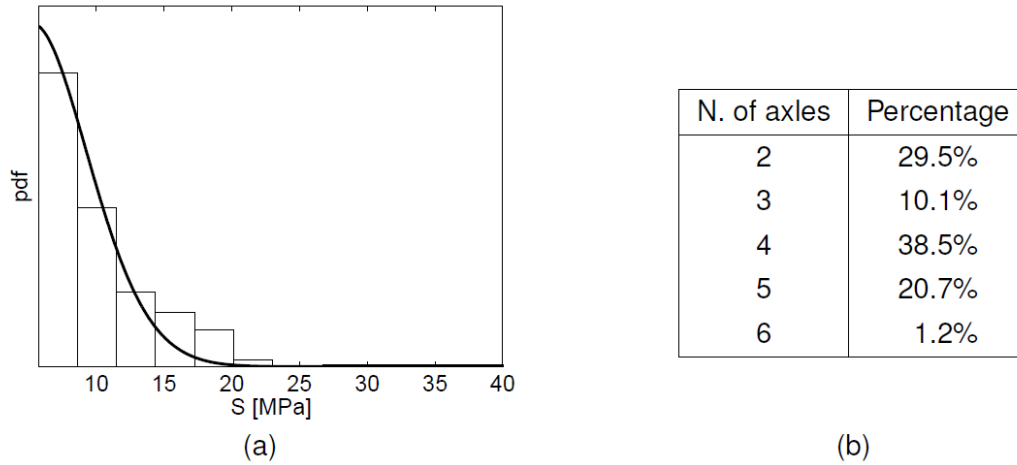


Fig. 79 Loading model: (a) Stress range spectrum; (b) Traffic composition according to 2006 WIM measurements.

The design value of allowed number of vehicles, n_{veh} , can be computed as follows:

$$n_{veh} = \frac{n_d}{\sum_{i=2}^6 i \cdot p_i} \quad (\text{Eq. 95})$$

where:

- i is the number of vehicle axles;
- p_i is the percentage of vehicles having i axles, according to Fig. 79 (b).

The comparison between the ML-MCS-based design value of n_{veh} and the Eurocode based design value of n_{veh} is presented in Fig. 80 for the two considered details.

For the cover-plate detail the ML-MCS approach provides slightly higher estimate of design value of n_{veh} with respect to the Eurocode approach. However, both approaches predict a fatigue design life higher than 100 years, for a traffic volume, $n_{veh/year} \leq 2 \cdot 10^6$.

For the in-plane gusset detail the ML-MCS approach provides considerably lower estimate of design value of n_{veh} with respect to the Eurocode approach. Fig. 81 shows the relationship between the design life in years and the number of crossing vehicles per year, $n_{veh/year}$, when the in-plane gusset is considered as the critical detail: it is noted that for $n_{veh/year} = 1.56 \cdot 10^6$, the Eurocode gives a design life equal to 100 years, which is about 60% of the design life predicted with ML-MCS approach. For $n_{veh/year} = 0.40 \cdot 10^6$ (which is the traffic volume measured by WIM on Venoge bridge in 2013, see Fig. 66) the predicted design life increases from 390 years (Eurocode) to 710 years (ML-MCS).

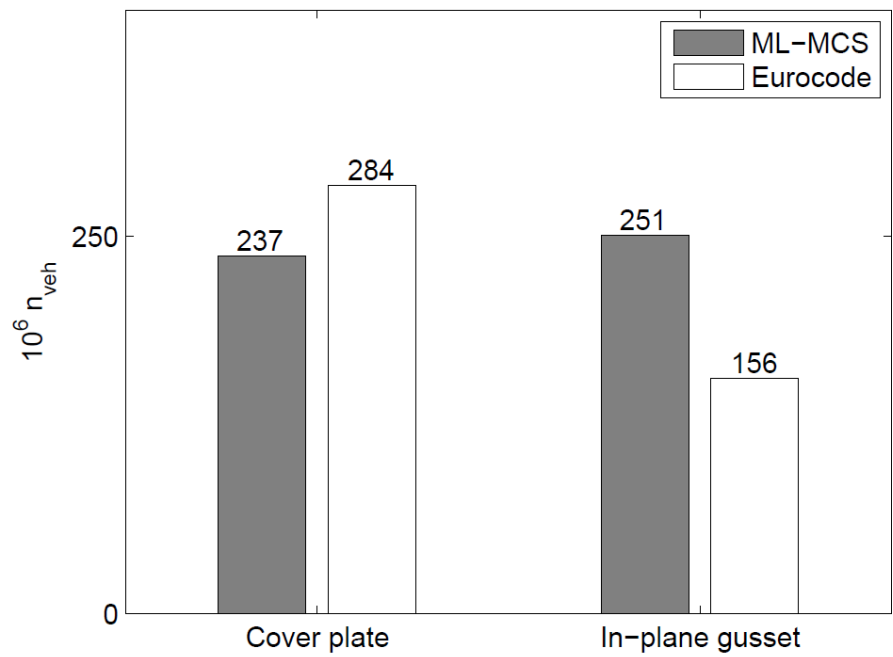


Fig. 80 Comparison of design values of n_{veh} for verification scheme 3.

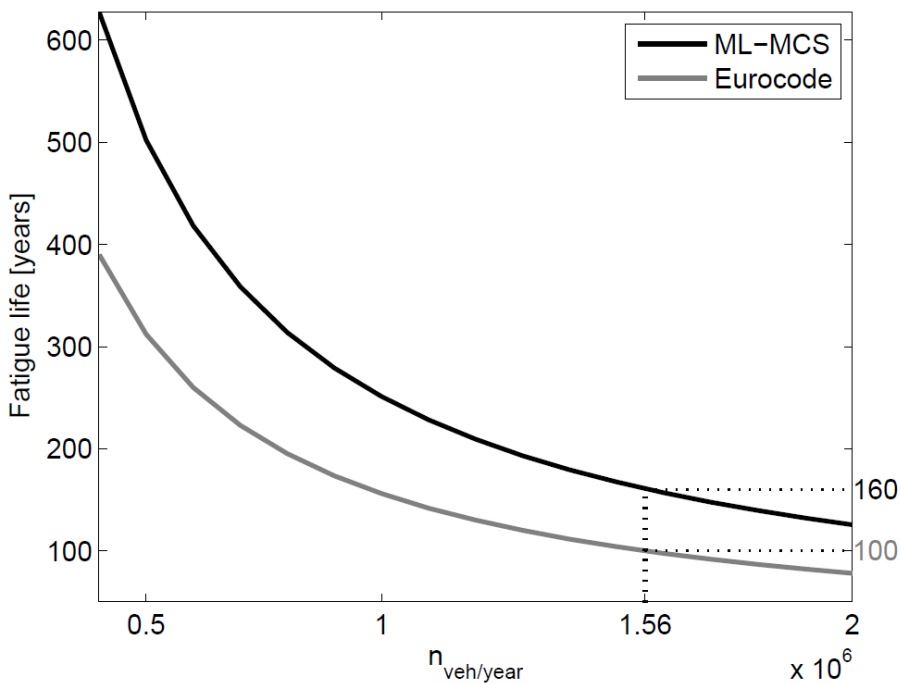


Fig. 81 Relationship between design fatigue life and traffic volume, in-plane gusset.

6.4 Structural evaluation indexes computation

The basis of the structural indexes has been presented in section 5.5. However, instead of one, the different expressions corresponding to the three fatigue verification schemes proposed in Eurocode standards are now given:

Verification scheme 1 : based on CAFL

$$\nu_1 = \frac{CAFL_d}{S_{max,d}} \quad (\text{Eq. 96})$$

Verification scheme 2 : based on equivalent stress range

$$\nu_2 = \frac{S_{c,d}}{S_{E,2,d}} \quad (\text{Eq. 97})$$

Verification scheme 3 : based on damage accumulation, which is here given with respect to traffic volumes

$$\nu_3 = \frac{n_{veh}}{\omega \cdot 100y}. \quad (\text{Eq. 98})$$

where:

- $CAFL_d$ is the design value of the CAFL;
- $S_{max,d}$ is the design value of the maximum stress range;
- $S_{c,d}$ is the design value of the fatigue strength at $2 \cdot 10^6$ cycles;
- $S_{E,2,d}$ is the design value of the equivalent stress range at $2 \cdot 10^6$ cycles;
- n_{veh} is the design number of allowed heavy vehicles, during 100 year bridge design life;
- ω is the observed traffic volume (in number of heavy vehicles per year).

The structural evaluation index, ν_3 was computed for the Venoge bridge study case, by using results of example presented in Section 6.3. The comparison between ML-MCS-based ν_3 and Eurocode-based ν_3 , for the two critical details (cover plate and in-plane gusset) and for different values of ω , are presented in Tab. 23:

Tab. 23 Comparison of structural evaluation indexes

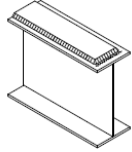
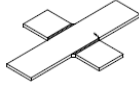
						
$\omega \rightarrow$	$0.4 \cdot 10^6$	$1.5 \cdot 10^6$	$2 \cdot 10^6$	$0.4 \cdot 10^6$	$1.5 \cdot 10^6$	$2 \cdot 10^6$
Eurocode	7.10	1.89	1.42	3.90	1.04	0.78
ML-MCS	5.93	1.58	1.19	6.28	1.67	1.26
	-17%			+61%		

Table 7.5 shows that:

- The values confirm what was found in Section 5.5, that is for the actual traffic, around $0.4 \cdot 10^6$ veh/year, the structural index is high, the bridge is very safe against fatigue ;
- Eurocode gives moderate under-conservative estimate of the index ν_3 with respect to the ML-MCS approach, for the cover-plate;
- Eurocode gives highly over-conservative estimate of the index ν_3 with respect to the ML-MCS approach, for the in-plane gusset. In particular, for a traffic volume $\omega = 2 \cdot 10^6$ veh/year, Eurocode approach gives a structural evaluation index lower than unity, while ML-MCS provides $\nu_3 = 1.26$. Note that the very aggressive and unrealistic virtual traffic scenarios presented in section 5.5 were not reused here.

6.5 Conclusions and discussion

This Chapter presented a new framework for calibration of partial safety factors used in fatigue design of steel bridges.

The presented framework improves consistency in achieving target levels of safety by:

- Defining one different design equation for each of three design verification criteria proposed in Eurocode standards; this allows to distinguish three different resistance partial safety factors, γ_{Mf} , to be used in: 1) Fatigue design based on CAFL; 2) Fatigue design based on CA equivalent stress range at $2 \cdot 10^6$ cycles; and 3) Fatigue design based on accumulated damage;
- Defining a method to assess which is the characteristic return period of the load effect which allows to set $\gamma_{Ef} = 1.0$ in the design equation;
- Using ML-MCS-based S-N curves, which results in a more realistic consideration of the CAFL and of the fatigue strength in HCF region;
- Using ML-MCS-based VA S-N models, which results in a more realistic consideration of damage accumulation.

The study cases considered in this Chapter show that:

- Calibration of partial safety factors for verification scheme 1 (based on CAFL) gives higher values of $\hat{\gamma}_{Mf}$ with respect to the verification scheme 2 (based on equivalent stress range at $2 \cdot 10^6$ cycles), for both details (see Tab. 21). This is probably due to the higher fatigue life scatter in the HCF region with respect to the scatter in the finite life region;

- The $\hat{\gamma}_{Mf}$ for the verification scheme 1 is considerably higher for the in-plane gusset ($\hat{\gamma}_{Mf} = 2.27$) with respect to the cover plate ($\hat{\gamma}_{Mf} = 1.80$). This is probably due to the more dispersed CAFL distribution of the in-plane gusset detail with respect to the cover plate detail (see Fig. 13 and Fig. 18);
- For all verification schemes the return period of characteristic load effect, $\hat{\rho}$, ranges between 500 and 1000 years (see Tab. 21);
- Eurocode standards give unsafe estimate of the design value of the CAFL, for both considered details (see Tab. 22). For the in-plane gusset detail this is due to the unsafe Eurocode-based calibration of resistance partial factor (see Tab. 19 and Tab. 21), while for the cover-plate gusset this is both due to the unsafe Eurocode-based calibration of resistance partial factor (see again Tab. 19 and Tab. 21) as well as to the unsafe Eurocode-based estimate of the characteristic value of the CAFL (see Section 3.6);
- Eurocode standards and ML-MCS approach give similar values of $\hat{\gamma}_{Mf}$ for the verification scheme 2 (see Tab. 19 and Tab. 21). The considerable difference on the design fatigue strength at $2 \cdot 10^6$ cycles (see Tab. 22), for the in-plane gusset detail, is due to the over-conservative Eurocode-based estimate of the characteristic fatigue strength (see Section 3.6);
- Fig. 77 shows that the Eurocode-based VA design S-N curve of the cover-plate detail is over-conservative with respect to the ML-MCS-based S-N curve for stress ranges higher than 19 MPa, while it becomes slightly under-conservative for stress ranges lower than 19 MPa. Fig. 78 shows that the Eurocode-based VA design S-N curve of the in-plane gusset detail is over-conservative with respect to the ML-MCS based S-N curve, at all stress ranges.

It is noted that the verification scheme 1 is the first mandatory step to carry out in fatigue design of new bridges (see Fig. 82): considered study cases show that verification scheme 1 including Eurocode-based design value of CAFL is highly un-safe.

In order to show the impact of the choice of design S-N curves on the fatigue design of a road bridge, fatigue life was computed for the Venoge bridge, by considering both Eurocode-based design S-N curves and ML-MCS-based design S-N curves. Results show that, for the critical in-plane gusset detail, Eurocode standards give a prediction of the allowed number of crossing vehicles ($156 \cdot 10^6$) which is considerably lower than the value predicted by the ML-MCS approach ($251 \cdot 10^6$). For a traffic volume of $\simeq 1.5 \cdot 10^6$ heavy vehicles per year, the design life increases from 100 years (Eurocode) to 160 years (ML-MCS).

The reliability framework for calibration of fatigue partial safety factors, presented and applied to two fatigue-sensitive details, constitutes a powerful tool that can be used to revise the Eurocode basis for fatigue design of structures. The three Eurocode formats for fatigue design and associated partial safety factors can be revised by considering different fatigue sensitive details and by further differentiating between: 1) Verification based on CAFL exceedance; 2) Verification based on equivalent stress range at $2 \cdot 10^6$ cycles, using lambda factors; and 3) Verification based on damage accumulation.

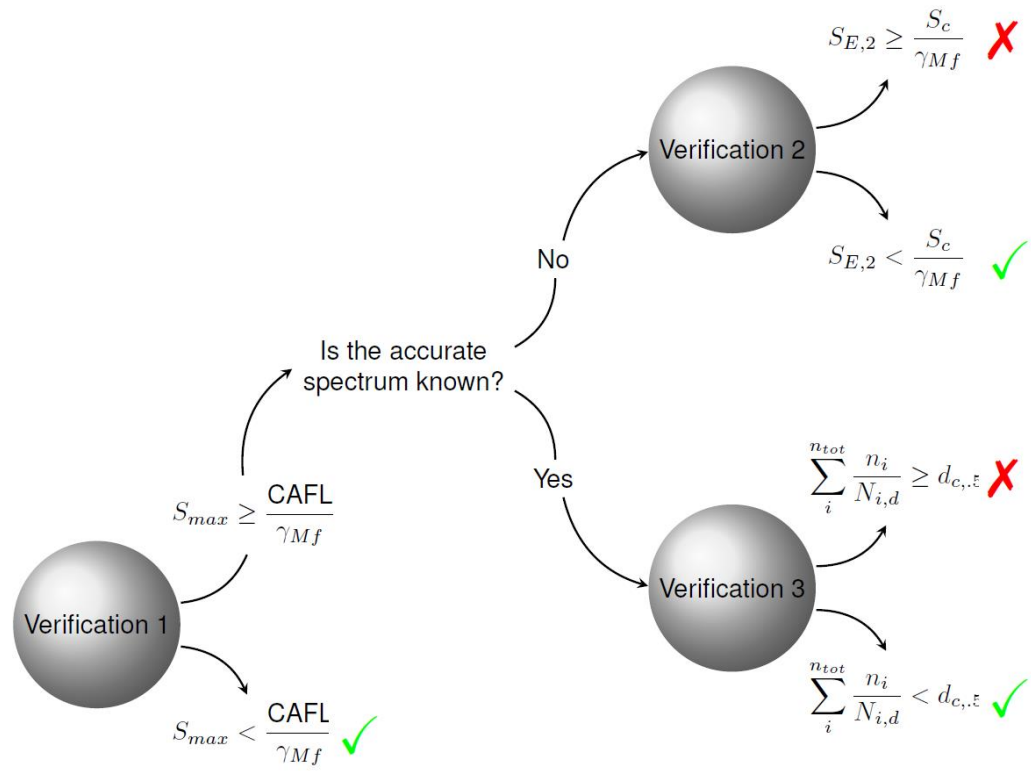


Fig. 82 Fatigue design verification chart.

7 Conclusions for part 2

7.1 Conclusions

In accordance with the objectives stated in Section 2.3, the main conclusions of this study are summarized as follows:

- The ML-MCS approach proposed for estimation of CA S-N curves constitutes a powerful tool for re-definition of standard CA S-N curves. The ML method was applied herein to welded joints. It allows to use run-out data for estimation of S-N curves. The novel idea of using MCS technique to estimate true p-quantiles of fatigue life gives more insight into realistic estimation of CAFL and knee point position for any detail.

According to considered study cases, arbitrarily fixing the CAFL at fixed number of cycles lead to an un-safe assumption which could be extremely risky (unsafe) for fatigue verifications based on CAFL.

It is recalled that the effectiveness of the proposed ML-MCS approach is conditioned to the use of experimental CA data-sets which have significant information in the HCF region. This issue is addressed in Section 7.2, future works;

- The ML-MCS approach proposed for estimation of VA S-N models constitutes a powerful tool for re-defining the second slope of S-N curves, m_2 , as well as the critical value of the damage sum used in Miner's rule, D_c . The novel idea of resimulating experimental VA fatigue tests and to estimate m_2 and D_c with ML method allows to: 1) Give experimental support to the amount of the change of S-N curve slope, $\Delta m = m_1 - m_2$; 2) Give experimental support to the choice of probability distribution of D_c .

The considered study cases show that the ML-MCS approach gives the most realistic estimation of failure damage sum with respect to existing standards. It is found that there is a strong influence of load sequence on D_c probability distribution; it follows that realistic load histories should be used for producing VA experimental results, when the proposed ML-MCS approach is used for re-definition of m_2 and D_c in current standards.

- The proposed approach for estimation of VA S-N models (and a fortiori CA S-N curves) can be used in improving standards for both new as well as existing structures. The fact is that the need for more accurate methods is important for existing structures, where assessment result in remaining fatigue lives values. Thus while re-definition of S-N curves in standards for assessment of existing bridges shall be discussed, the current Eurocode S-N curves (new structures) can be kept at this time.
- The reliability analysis framework set up allows for performing fatigue reliability assessment of existing road steel bridges using WIM-based characterization of traffic load history. For specific projects, the use of ML-MCS-based S-N curves is needed as it improves the confidence in fatigue life estimations using fatigue reliability analysis. The novel contribution of the proposed framework consists in computing the failure probability as joint probability of: 1) The CAFL exceedance event; and 2) The critical damage accumulation event.

The considered Venoge bridge study case, including a study with consideration of two virtual traffic evolution scenarios, shows the inaccuracy of Eurocode approach in assessment of reliability index during the 100 year-design life of the bridge ;

- The new framework set up for calibration of fatigue partial safety factors allows for revising the Eurocode basis for fatigue design of new bridges. Use of ML-MCS-based VA S-N models gives more insights into realistic consideration of fatigue strength in HCF region as well as damage accumulation mechanism. New method is proposed to define the characteristic return period of load effect which allows to set the load partial

factor, γ_{Ff} equal to unity in the design equation. Revision of resistance partial safety factor, γ_{Mf} , can be performed by differentiating between three verification schemes:

1) Based on CAFL exceedance; 2) Based on equivalent stress range at $2 \cdot 10^6$ cycles, using lambda factors; and 3) Based on damage accumulation.

Considered study cases do not support the Eurocode assumption of considering the same γ_{Mf} for all three verification schemes. Eurocode standards give highly unsafe estimate of design CAFL, for both study cases: fatigue design based on verification scheme 1, using Eurocode-based γ_{Mf} , should thus not be used as it could lead to unsafe designs.

It is recalled here that the verification scheme 1 is the first mandatory step in fatigue design of new bridges, which can be followed by schemes 2 or 3 according to the choice of the engineer (see Fig. 82).

More detailed discussion on specific results of each Chapter as well as numerical results of considered study cases are presented at the end of each Chapter.

7.2 Future works

The results of this study provide a strong foundation for future work in fatigue verification of road steel bridges. The study has shown that there is a need of more CA and VA fatigue experimental results in the HCF region. Four areas of future work have been identified as follows:

- Re-definition of CA S-N curves in future Eurocode standards for existing structures;
- Re-definition of Miner's rule in future Eurocode standards for existing structures;
- Revision of Eurocode formats, both for new and existing structures, for fatigue design and re-calibration of associated partial safety factors;
- Inclusion of the bi-linear CA S-N model without CAFL into the ML-MCS approach.

The following Section proposes future work in these four areas.

Re-definition of CA S-N curves

In order to re-define CA S-N curves, in particular of welded joints, in future Eurocode standards for existing structures, a fatigue test program should be carried out according to following steps:

1. Choice of details to include in the re-definition (i.e. longitudinal attachment, transversal attachment, cover plate, T-joint, . . .);
2. CA fatigue testing of selected details. It is recommended to use stress ranges varying from $1.5 \cdot \text{FAT}_{\text{EN}}$ to $0.5 \cdot \text{CAFL}_{\text{EN}}$, where FAT_{EN} and CAFL_{EN} are respectively the FAT and the CAFL of tested details, according to EN 1993-1-9. A minimum number of three data points is required for each stress range level. Experimental tests in which $N = 10^8$ will be reached without failure will be classified as run-outs. Use of the scheme recommended above will ensure to have significant CA experimental results in the HCF region;
3. Estimation of CA S-N stochastic models as well as characteristic S-N curves of considered details, using ML-MCS approach. Matlab Toolbox TB1 (see Appendix III) should be used.

Re-definition of Miner's rule

Once the CA S-N stochastic model has been estimated for the selected details, a VA fatigue test program needs to be carried out in order to re-define S-N curve second slope, m_2 , and critical damage, D_c , in future Eurocode standards for existing structures. Following steps should be followed:

4. VA fatigue testing of selected details. Due to the strong influence of load history on the critical damage sum D_c , it is recommended to use realistic loading spectra for VA tests; one possibility would be to repeatedly apply real road bridge recorded load histories to the tested specimens.

It is recommended to use a number of cycles, $\tilde{n}_{tot}$, for each loading spectrum, such that:

$$\int_{\exp(\hat{\mu}_V)}^{s_{max}} \frac{\tilde{n}_{tot} \cdot \tilde{f}_S(s)}{N(s)} ds \leq 3.0 \quad (\text{Eq. 99})$$

where $\tilde{f}_S(s)$ is the loading spectrum, s_{max} is the maximum stress range of the loading spectrum, $\hat{\mu}_V$ is the ML estimate of median value of log-CAFL and $N(s)$ is the number of cycles to failure according to median CA S-N curve²⁷. Experimental tests in which n_{tot} will be reached without failure will be classified as run-outs. Use of the scheme recommended above will ensure to have significant VA experimental results in the HCF region.

It is recommended to use stress range loading spectra which have ζ varying from 0.5 to 0.05, where ζ is the percentage of stress range cycles exceeding the characteristic value of the ML-MCS based CAFL. A minimum number of five different load spectra should be used and a minimum number of three data points is required for each stress range loading spectrum;

Estimation of VA S-N stochastic models of considered details, which include $m_2 = (m_1 - \Delta m)$ and D_c , using ML-MCS approach. Matlab Toolbox TB1 (see Appendix III) should be used.

Revision of Eurocode formats for fatigue design and re-calibration of associated partial safety factors

Once the CA and VA S-N stochastic models of selected details have been estimated, the three Eurocode formats for fatigue design can be revised using the framework developed in Chapter 6. Matlab Toolbox TB3 (see Appendix III) should be used to re-calibrate partial resistance factors associated to the three verification formats.

A further improvement may be introduced in the developed framework, by considering realistic loading spectra instead of simple Rayleigh loading spectra.

Inclusion of the bi-linear CA S-N model without CAFL into the ML-MCS approach

The ML-MCS approach presented in this work is based on linear CA S-N curve having random CAFL. The existence of CAFL is still a topic for debate; Sonsino [75] suggests that CAFL does not exist and that stress range cycles below the knee point should be accounted with a slope $m_1' = -22$. The knee point is arbitrarily fixed at 10^7 cycles; due to the lack of experimental results in the HCF region, $m_1' = -22$ is arbitrarily chosen in order to have a 10% constant stress range decrease with respect to the $\log(N)$ axis. The ML-MCS approach presented in this report can be adapted to a bi-linear S-N model

²⁷ According to results presented in Chapter 4, $d_c = 3.0$ is higher than the 0.9 quantile of D_c distribution for both cover-plate and in-plane gusset study cases: the choice of $d_c = 3.0$ in Equation 99 will then ensure to have a significant run-out if the experimental test ends without a failure after $\tilde{n}_{tot}$ cycles.

having a slope $m_1' = -22$ for stress range cycles below the knee point. The bi-linear model should be defined as follows:

$$Y = \begin{cases} m_0 + m_1 X + \epsilon(0, \exp(\sigma)), & \text{for } X > X_k \\ m_0 + \mu_{X_k}(22 + m_1) - 22X + \epsilon(0, 22 \cdot \exp(\sigma_{X_k})), & \text{for } X \leq X_k \end{cases} \quad (\text{Eq. 100})$$

where μ_{X_k} and $\exp(\sigma_{X_k})$ are respectively the location and the scale parameters of the X_k random variable, which represents the natural logarithm of the stress range at the knee point (see Fig. 83).

Thereafter, the model parameter vector, $\underline{\theta}$, relative to the S-N model with random CAFL, should be transformed as follows:

$$\underline{\theta} = (m_0, m_1, \sigma, \mu_V, \sigma_V) \rightarrow \underline{\theta} = (m_0, m_1, \sigma, \mu_{X_k}, \sigma_{X_k}) \quad (\text{Eq. 101})$$

As for the case of the S-N model with CAFL, the ML-MCS approach has the advantage of estimating the knee point instead of arbitrarily assuming its position.

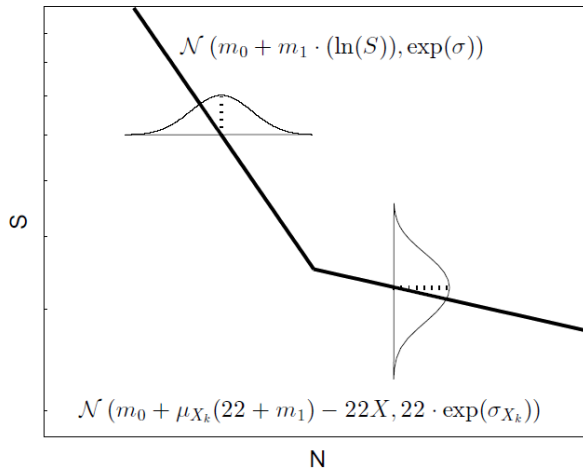


Fig. 83 Bi-linear S-N curve without CAFL.

Future work is required for: 1) Implementing the CA bi-linear S-N model into the ML-MCS approach; and 2) Validating it with experimental data-sets which contain significant information in the HCF region.

8 General conclusions

In order to define a road traffic model, this research shows the necessity to carry out local studies instead of drawing upon a model applied at national level in terms of traffic composition and loads. The results found and proposed approaches allow achieving, in function of the desired degree of detail, and adapted to each studied case, a traffic load model and improved safety checks.

Regarding heavy vehicles traffic loads, the research is based on statistical investigations of road counting and weight in motion stations (WIM), on studies of videos as well as accidentology, at different time scales (day to years). It has allowed to determine a behavioural model for heavy vehicles. Thanks to this model, which has to be adapted to each specific situation, it is then possible to evaluate in an accurate way the effects of total traffic (light and heavy vehicles), being it in the past, present or future by performing traffic simulations on a given duration (usually one average week).

In order to demonstrate the approach, it is applied to three study cases :

- Denges, highway A1, near the Venoge bridge on the Lausanne - Genève section (the whole network with the nodes has been developed : Mex – Maladière – Ecublens – Morges – Genève airport)
- Monte Ceneri, highway A2, along section Lugano – Bellinzona
- Aarwangen, bridge on the Aare, cantonal road n°244 Langenthal – Niederbipp-Oensingen

The comparisons between traffic simulations and real traffic demonstrate the excellent quality of the simulations in terms of flows, average speeds and fluctuations over 10 replicates. In addition, they confirm a median heavy vehicle speed of 88 km/h, higher than the official limit and however be an explanation for a potential reduction of implication of heavy vehicles in accidents.

This project has allowed to create interfaces between the different existing tools and thus the use of the traffic simulations results for the assessment of existing bridges and pavement degradation subjected to traffic loads via evaluation indexes *v.* For pavement degradation, two postulates allowing for the determination of evaluation index are given. For fatigue, this is done thru the interfaces between AIMSUN (traffic simulation software) and Matlab routines in order to determine the forces in the controlling sections/details.

In order to estimate fatigue strength curves needed for the fatigue evaluation index computation, the combined approach « maximum likelihood » (ML) and Monte Carlo simulations (MCS) proved to be a powerful tool to redefine standard S-N curves (characteristic curves). With the p-fractiles estimates from MCS, a more realistic evaluation of the position of the knee point and fatigue limit (CAFL) is possible. This allows to conclude that for the studied details, in-plane gusset and cover plate, the fixed position at 5 millions cycles for the knee point, which defines the CAFL value, leads to unsafe fatigue checks with respect to that same CAFL. The same approach, further developed, is also a powerful tool to redefine the variable amplitude part (second slope, m_2) of the standardized S-N curves, as well as a more realistic evaluation of the critical value of the damage sum, D_c . It is important to insist on the fact that the efficiency of the proposed approach, and trust in the fatigue check, is conditioned by the results available containing enough information (under CA and VA) in the region of high cycle fatigue ($N > 5 \cdot 10^6$ cycles), which actually is often missing.

A new methodology developed in this study with the ML-MCS approach further allows to estimate the fatigue reliability of existing bridges by using simulated traffic. In the new framework the probability of fatigue failure is defined by taking into account both damage accumulation and CAFL exceedance critical events. Finally, the ML-MCS approach was also integrated in a new framework for re-calibration of fatigue design partial safety factors for structures (new or existing). Applied to two code fatigue details, it again

showed that fatigue checks with respect to CAFL are unsafe (with current Eurocode γ_{Mf} values). Even if applied to welded joints, all developed tools related to fatigue have general applicability. For comparison, fatigue reliability estimation was applied to the study case of the existing Venoge bridge. While the bridge has a high reliability index, the methodology demonstrates the inaccuracy of the Eurocode check to estimate the reliability factor evolution during the bridge life of 100 years.

The proposed approach for estimation of VA S-N models (and a fortiori CA S-N curves) can be used in improving standards for both new as well as existing structures. The fact is that the need for more accurate methods is important for existing structures, where assessment result in remaining fatigue lives values. Thus while re-definition of S-N curves in standards for assessment of existing bridges shall be discussed, the current Eurocode S-N curves (new structures) can be kept at this time.

More complete discussion of the conclusions presented above can be found in the chapter preceding this one of the each of the reports for part 1 (traffic) or part 2 (fatigue), according to the topic.

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I Nomenclature and definitions

a	Crack length
$A(t)$	Crossing axle time history
C	Intercept of generic CA S-N curve in log(S-N) plane (material constant)
$\underline{c}$	CA and VA S-N model correlation matrix
CA	Constant Amplitude
CAFL	Constant Amplitude Fatigue Limit
cdf	Cumulative distribution function
c_v	Coefficient of variation
C_1	Proportionality constant in Paris Law
D	Damage sum at failure
d	Observed damage sum at the end of the experimental test
D_c	Critical damage sum (random variable)
$d_{c,5}$	Characteristic value of critical damage sum, corresponding to the median of D_c
D_d	Accumulated damage over the bridge design life
E_c	Characteristic value of action effect
E_1	Critical damage accumulation event
E_1^*	Critical damage accumulation conditioned to CAFL exceeding (conditioned event)
E_2	CAFL exceeding event
EN3	Eurocode 3
EV	Extreme Value probability distribution
FAT	Fatigue strength in MPa at $2 \cdot 10^6$ cycles, according to characteristic S-N curve
FE	Finite elements
FL	Finite life: $N < 5 \cdot 10^6$ cycles, convention according to Eurocode
FLS	Fatigue limit state
FORM	First order reliability method

f_Y	Probability density function of random variable Y
F_Y	Cumulative distribution function of random variable Y
$\mathcal{G}$	Gumbel distribution
GEV	Generalized Extreme Value probability distribution
GTW	Gross Total Weight
$g(\underline{X})$	Limit state function for basic random vector $\underline{X}$
$H(\cdot)$	Unit step function
HCF	High cycle fatigue: $N \geq 5 \cdot 10^6$ cycles
$\underline{\underline{I}}(\underline{\theta})$	Fisher information matrix for parameter vector equal to $\underline{\theta}$
k	Shape parameter of Weibull stress range loading spectrum. The spectrum is concave downward for $k < 1$ and it is concave upward for $k > 1$.
knee point	In a CA S-N curve it is the intersection point between the S-N curve and the CAFL; in a VA S-N curve it is the intersection point between the S-N curve having slope equal to m_1 and the S-N curve having slope m_2
K	Stress intensity factor
log(S-N) plane	S-N plane in which both S axis and N axis are plotted in log scale
$L(\underline{\Theta})$	Sample likelihood for model parameter vector equal to $\underline{\theta}$
LEFM	Linear Elastic Fracture Mechanics
LS	Least Squares
m	Slope of generic CA S-N curve in log(S-N) plane (< 0)
MCS	Monte-Carlo simulations
ML	Maximum Likelihood
M_k	Geometric correction factor in Paris Law
m_0	Intercept of the CA S-N curve, in the log(S-N) plane
m_1	Slope of the CA S-N curve, in the log(S-N) plane
m'_1	Slope of the CA S-N curve below the knee point, in the log(S-N) plane, for CA S-N bi-linear model without CAFL
m_2	Slope of the VA S-N curve below the CAFL, in the log(S-N) plane
n	Number of stress range cycles applied to the analysed detail
N	Number of stress range cycles to the end of the test
$\log \mathcal{N}$	Log-Normal probability distribution

$\mathcal{N}$	Normal probability distribution
n_d	Design value of allowed number of cycles which can be applied to analysed detail
n_f	Number of failures
n_r	Number of run-outs
$nLL(\underline{\theta})$	Negative log-likelihood for model parameter vector equal to $\underline{\theta}$
n_{veh}	Design value of allowed number of vehicles
n_1	Number of cycles applied to the analysed detail, with stress range higher than the CAFL
n_2	Number of cycles applied to the analysed detail, with stress range lower than the CAFL
$\mathcal{O}$	Objective function in MCS scheme for estimation of CA characteristic S-N curves
pdf	Probability density function
P_f	Probability of failure
P_f^t	Target probability of failure
PLR	Profile Likelihood Ratio
p -quantile	Referred to a S-N curve, it indicates the curve which realizes $p \cdot 100\%$ probability of failure
$P_s = 1 - P_f$	Probability of survival
R	Stress ratio
R_c	Characteristic value of resistance
$R_d(\mu_{R_d}, \sigma_{R_d})$	Sum of m-power stress ranges over design life (random variable)
RFL RMC	Root mean cube Random Fatigue Limit
r.m.s	Root mean square of stress range spectrum: $\sqrt{\sum (S_i/n)}$
run-out	Number of cycles reached without failure of the specimen at the end of the test, either because the time limits for the test were reached or because other parts of the tested specimen failed
$R_w(\mu_{R_w}, \sigma_{R_w})$	Weekly sum of m-power stress ranges (random variable)
S	Nominal applied stress range
S	Sample standard deviation
S_c	Characteristic fatigue strength at $2 \cdot 10^6$ cycles

S_{co}	Cut-off stress range
$S_{E,2}$	Equivalent CA stress range at $2 \cdot 10^6$ cycles
S_{max}	Maximum stress range
$S_{max,c}$	Characteristic value of maximum stress range
S_n	Nominal stress
SORM	Second order reliability method
$t_{\alpha,n}$	Inverse of Student's t cdf for probability equal to $\alpha\%$ and n degrees of freedom
V	Natural logarithm of the CAFL (random variable)
VA	Variable Amplitude
$v_{.05}$	Characteristic value of the log-CAFL, corresponding to 95% exceeding probability
w	Importance factor
$\mathcal{W}$	Weibull distribution
WIM	Weigh in motion
$W(\gamma)$	Objective function in framework for calibration of partial safety factors
x_c	Natural logarithm of characteristic fatigue strength, at $2 \cdot 10^6$ cycles
$x_{c,LS}$	LS estimate of fatigue strength at $2 \cdot 10^6$ cycles
X_k	Natural logarithm of the stress range at the knee point
$(x_i, y_i) \mid_{i=1 \dots n_{tot}}$	CA fatigue data-set
z	Design factor in fatigue design and limit state equations
$z_p(\tau, \phi, \xi)$	Return level of maximum stress range, following a $GEV(\tau, \phi, \xi)$ probability distribution
α	Confidence level
β	Reliability index associated to the fatigue failure event
β_t	Target reliability index
β_1	Reliability index associated to the critical damage accumulation event
β_2	Reliability index associated to the CAFL exceeding event
γ_F	Partial safety factor for loading
γ_{Ff}	Fatigue partial safety factor for loading
γ_M	Partial safety factor for resistance

γ_{Mf}	Fatigue partial safety factor for resistance
δ	Binary variable which is equal to 1 for a failure and to 0 for a run-out
$\Delta m = m_1 - m_2$	Slope difference, in the log(S-N) plane
ΔK	Difference between the stress intensity factor at the upper and lower limit stresses of the loading cycle
ϵ	Error term in S-N model
η	Number of simulations
η_F	Number of simulations which realize failure condition
η_{iter}, η_{res}	Parameters which define the number of simulations in ML-MCS scheme for estimation of VA S-N model
η_{sam}	Number of simulations in MCS scheme for estimation of CA characteristic S-N curves
θ	CA S-N model vector
Θ	VA S-N model vector
λ_R	Scale parameter of Rayleigh stress range loading spectrum
μ_D	Location parameter of critical damage probability distribution, D_c
μ_V	Location parameter of log-CAFL probability distribution
σ	Logarithm of the scale parameter of log-life probability distribution
σ_V	Logarithm of the scale parameter of log-CAFL probability distribution
σ_D	Scale parameter of critical damage probability distribution, D_c
$\underline{\underline{\Sigma}}$	VA S-N model covariance matrix
$\underline{\underline{\rho}}$	CA S-N model correlation matrix
$\rho_{S_{max}}, \rho_{S_{E2}}, \rho_{n_{tot}}$	Characteristic return periods of S_{max} , S_{E2} and n_{tot}
ϕ	Standardized normal pdf
ϕ_d	Load dynamic amplification factor
Φ	Standardized normal cdf
$(\varpi_t, y_t) _{t=1 \dots n_{ts}}$	VA fatigue data-set
ς	Percentage of stress range cycles of a loading spectrum, which exceeds the characteristic value of CAFL

II Confidence and prediction bounds for fatigue

II.1 Confidence intervals

In parameter inference, an interval estimator is a rule which specifies the method that we have to use to compute the bounds of the interval which will contain the target parameter. Ideally, the interval will have two properties: 1) It will contain the target parameter, θ ; 2) It will be relatively narrow. The bounds of the interval are computed from the sample measurements and will vary randomly from sample to sample; hence, the location and the length of the interval is a random quantity and we can not be certain that the target parameter, θ , will fall into the interval computed from one single sample. Interval estimators are commonly called *confidence intervals*; the probability that θ will fall into a random confidence interval is called confidence level. The confidence level gives the fraction of time, in repeated sampling, that the computed confidence intervals will contain θ .

Let θ_l and θ_u denote the random lower and upper confidence interval bounds, respectively for the parameter, θ .

Then, if:

$$P(\theta_l \leq \theta \leq \theta_u) = 1 - \alpha \quad (\text{Eq. 1})$$

the probability $(1 - \alpha)$ is the confidence level associated with the confidence interval $[\theta_l, \theta_u]$.

II.2 Large sample confidence intervals

II.2.1 Confidence interval for the mean

Let $(Y_1, \dots, Y_n)$ be a random sample of size n from a normal distributions with mean, μ and standard deviation, σ .

Then the sample mean:

$$\bar{Y} = \frac{1}{n} \sum_{i=1}^n Y_i \quad (\text{Eq. 2})$$

is normally distributed with mean, $\mu_{\bar{Y}}$ and standard deviation, $\sigma_{\bar{Y}} = \sigma / \sqrt{n}$.

Then the random variable:

$$Z = \frac{\bar{Y} - \mu_{\bar{Y}}}{\sigma_{\bar{Y}}} = \frac{\bar{Y} - \mu}{\sigma / \sqrt{n}} \quad (\text{Eq. 3})$$

has a standardized normal distribution, with:

$$P\left(\bar{Y} + z_{\frac{\alpha}{2}} \frac{\sigma}{\sqrt{n}} \leq \mu \leq \bar{Y} + z_{\frac{1-\alpha}{2}} \frac{\sigma}{\sqrt{n}}\right) = 1 - \alpha \quad (\text{Eq. 4})$$

The $(1-\alpha) \cdot 100\%$ confidence interval for μ is:

$$\left(\bar{Y} + z_{\frac{\alpha}{2}} \frac{\sigma}{\sqrt{n}}, \bar{Y} + z_{\frac{1-\alpha}{2}} \frac{\sigma}{\sqrt{n}}\right) \quad (\text{Eq. 5})$$

If the parameter σ is unknown we can substitute σ for the sample standard deviation, $\hat{\sigma}$:

$$\hat{\sigma} = \frac{1}{n-1} \sum_{i=1}^n (Y_i - \bar{Y})^2 \quad (\text{Eq. 6})$$

In this case the random variable:

$$T = \frac{\bar{Y} - \mu}{\hat{\sigma}/\sqrt{n}} \quad (\text{Eq. 7})$$

has a Student's T distribution (see Fig. 1) with n degrees of freedom and the $(1-\alpha) \cdot 100\%$ confidence interval for μ is:

$$\left(\bar{Y} + t_{\frac{\alpha}{2}} \frac{\hat{\sigma}}{\sqrt{n}}, \bar{Y} + t_{\frac{1-\alpha}{2}} \frac{\hat{\sigma}}{\sqrt{n}}\right) \quad (\text{Eq. 8})$$

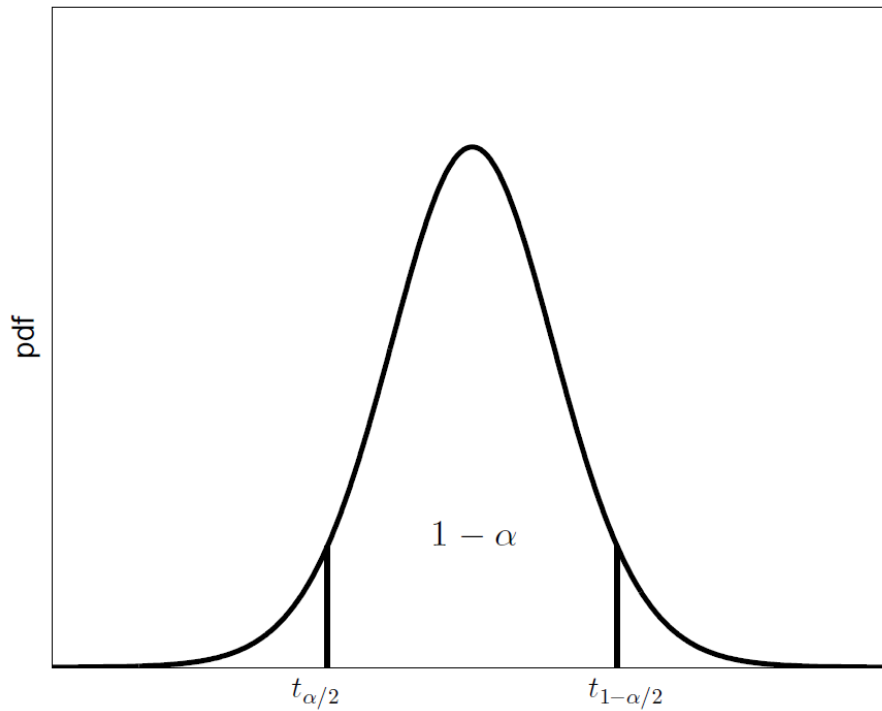


Fig. 1 Student's T probability density function.

II.2.2 Confidence interval for the standard deviation

Let $(Y_1, \dots, Y_n)$ be a random sample of size n from a normal distribution with mean, μ , and standard deviation, σ .

Then the random variable:

$$\frac{(n-1)\hat{\sigma}^2}{\sigma^2} = \frac{1}{\sigma^2} \sum_{i=1}^n (Y_i - \bar{Y})^2 \quad (\text{Eq. 9})$$

has a Chi-squared distribution with $n-1$ degrees of freedom (see Fig. 2). The $(1-\alpha) \cdot 100\%$ confidence interval for σ is:

$$\left(\frac{(n-1)\hat{\sigma}^2}{\chi_{1-\frac{\alpha}{2}, n-1}^2}, \frac{(n-1)\hat{\sigma}^2}{\chi_{\frac{\alpha}{2}, n-1}^2} \right) \quad (\text{Eq. 10})$$

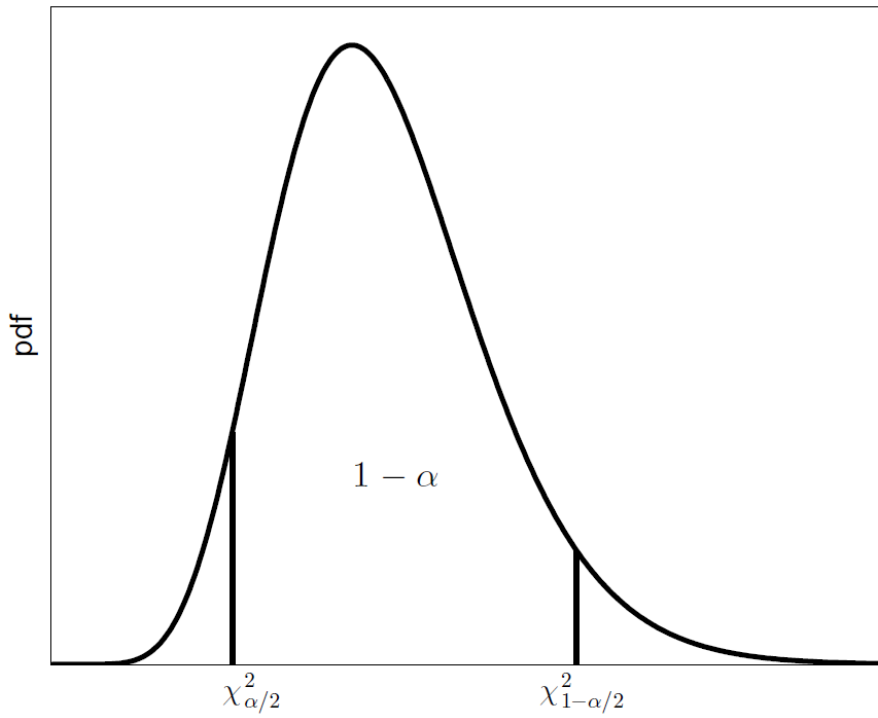


Fig. 2 Chi-squared probability density function.

I.2.2.1. The central limit theorem

The results given in Section II.2 are valid if $(Y_1, \dots, Y_n)$ is a random sample of size n from a Normal distribution; what happens if random variables $(Y_1, \dots, Y_n)$ are not normally distributed?

We can invoke the *central limit theorem* (see [84], p. 370). Let $(Y_1, \dots, Y_n)$ be independent and identically distributed random variables, with $E(Y_i) = \mu$, and $\text{Var}(Y_i) = \sigma^2$.

If we define the random variable:

$$Y^* = \frac{\bar{Y} - \mu}{\frac{\sigma}{\sqrt{n}}} \quad (\text{Eq. 11})$$

the distribution function of Y^* converges to the standard Normal distribution as $n \rightarrow \infty$ (usually $n > 30$ is accepted). In that case results shown in Section II.2 remain valid.

II.3 Confidence intervals of S-N curves - IIW approach

S-N curves are estimated using experimental results from CA fatigue experimental tests. Each experimental test is represented by a couple $(x_i = \ln(S_i), y_i = \ln(N_i))$, where S_i is the nominal applied stress range and N_i is the number of cycles until the failure (or run-out if the test has been stopped without failure). Within the assumption that the logarithm of number of cycles $Y(X)$ is a Normal random variable, then each experimental result $y_i(x_i)$ can be seen as a realization of the Normal random variable $Y(X)$.

The information contained in the experimental dataset can be used to make inferences about the random variable $Y(X)$ from which the sample is taken. The Normal random variable $Y(X)$ is defined by the location parameter, μ_Y , and by the scale parameter, σ_Y .

According to IIW recommendations [42] the linear model:

$$Y(x) = m_0 + m_1 x + \varepsilon(0, \sigma) \quad (\text{Eq. 12})$$

represents the response function traced by Y over the experimental region of interest.

The sample mean:

$$\hat{y}(x) = \hat{m}_0 + \hat{m}_1 x \quad (\text{Eq. 13})$$

is a point estimator of $\mu_Y(x)$ and can be computed using a least-squares (LS) approach: the sum of differences between the observed y_i and the predicted $(\hat{m}_0 + \hat{m}_1 x_i)$ is minimized.

Only the failure points are considered.

The LS unbiased estimators of model parameters are:

$$\hat{m}_1 = \frac{n \cdot \sum_{i=1}^n (x_i \cdot y_i) - \sum_{i=1}^n x_i \cdot \sum_{i=1}^n y_i}{n \cdot \sum_{i=1}^n (x_i)^2 - \left(\sum_{i=1}^n x_i \right)^2} \quad (\text{Eq. 14})$$

$$\hat{m}_0 = \bar{y} - \hat{m}_1 \bar{x} \quad (\text{Eq. 15})$$

The sample standard deviation:

$$\hat{\sigma} = \sqrt{\frac{1}{n-2} \sum_{i=1}^n (y_i - \hat{y}(x_i))^2} \quad (\text{Eq. 16})$$

is an unbiased point estimator of σ_Y and is independent of both $\hat{m}_0$ and $\hat{m}_1$, under the assumption of normality of error term, $\varepsilon(0, \sigma)$ (see p.582 in [84]).

As shown in Section II.2.1, the 75% two-side confidence interval for $\mu_Y(x)$ is:

$$\left(\hat{y}(x) + t_{.125, n-2} \frac{\hat{\sigma}}{\sqrt{n}}, \hat{y}(x) + t_{.875, n-2} \frac{\hat{\sigma}}{\sqrt{n}} \right) \quad (\text{Eq. 17})$$

Note that Student's T distribution with $n-2$ degrees of freedom is used since the mean function is defined using two independent parameters, m_0 , and m_1 .

As shown in Section II.2.1, the 75% two-side confidence interval for σ_Y^2 is:

$$\left(\frac{(n-2)\hat{\sigma}^2}{\chi_{.875, n-2}^2}, \frac{(n-2)\hat{\sigma}^2}{\chi_{.125, n-2}^2} \right) \quad (\text{Eq. 18})$$

Note that the Chi-squared distribution with $n-2$ degrees of freedom is used.

According to IIW recommendations the characteristic S-N curve is obtained by translating the median S-N curve (coincident with mean S-N curve) at the .05 quantile of the $Y(x)$ Normal distribution, by using the lower bound of the 75% two-side confidence interval of $\mu_Y(x)$ and the upper bound of the 75% two-side confidence interval of σ_Y .

The equation of the characteristic S-N curve is then:

$$y = \hat{m}_0 + \left(\frac{t_{.125, n-2}}{\sqrt{n}} + -1.645 \sqrt{\frac{n-2}{\chi_{.125, n-2}^2}} \right) \cdot \hat{\sigma} + \hat{m}_1 x \quad (\text{Eq. 19})$$

The issue of the estimation of the CAFL position is not addressed.

II.4 Prediction intervals

Let's consider the linear model expressed by the Equation 12. In the previous section we discussed the method to estimate confidence intervals for μ_Y and σ_Y .

Now we discuss a different problem: the distribution of the random variable $Y(x) = \mathcal{N}(m_0 + m_1 x, \sigma)$ being known, we want to predict the particular response $Y(x)$ that we will observe if the experiment is run at some time in the future; we are interested in predicting the value of Y when $x = x^*$, called Y^* . It is possible to show (proof is omitted in this document) that the interval:

$$\left(\hat{m}_0 + \hat{m}_1 x^* + t_{\frac{\alpha}{2}, n-2} \hat{\sigma} \cdot \sqrt{1 + \frac{1}{n} + \frac{(x^* - \bar{x})^2}{\sum_{i=1}^n (x_i - \bar{x})^2}}, \hat{m}_0 + \hat{m}_1 x^* + t_{1-\frac{\alpha}{2}, n-2} \hat{\sigma} \cdot \sqrt{1 + \frac{1}{n} + \frac{(x^* - \bar{x})^2}{\sum_{i=1}^n (x_i - \bar{x})^2}} \right) \quad (\text{Eq. 20})$$

in repeated sampling will contain the actual value of Y^* with probability $1 - \alpha$. Equation 20 represents a $100 \cdot (1 - \alpha)\%$ hyperbolic prediction interval for Y^* .

II.5 Prediction intervals of S-N curves - Eurocode approach

Statistical method for assessment of mean S-N curve, that is discussed in section II.3, is used again in Commentary to Eurocode 3, part 1-9 document [74].

The characteristic S-N curve is then obtained by translating the median S-N curve at the 5% lower hyperbolic prediction bound of Y , in correspondence of $N = 2 \cdot 10^6$ cycles:

$$\hat{m}_0 + \hat{m}_1 x_{cLS} + t_{.05, n-2} \cdot \hat{\sigma} \cdot \sqrt{1 + \frac{1}{n} + \frac{(x_{cLS} - \bar{x})^2}{\sum_{i=1}^n (x_i - \bar{x})^2}} \quad (\text{Eq. 21})$$

where x_{cLS} is the LS estimate of log-strength at $2 \cdot 10^6$ cycles.

The equation of the characteristic S-N curve is then:

$$y = \hat{m}_0 + t_{.05, n-2} \cdot \hat{\sigma} \cdot \sqrt{1 + \frac{1}{n} + \frac{(x_{cLS} - \bar{x})^2}{\sum_{i=1}^n (x_i - \bar{x})^2}} + \hat{m}_1 x \quad (\text{Eq. 22})$$

II.6 Discussion

Two different statistical methods for assessment of characteristic S-N curves are recommended from IIW and Eurocode. According to the Commentary to Eurocode 3, part 1-9 [74], the characteristic S-N curve is obtained by translating the median S-N curve at the .05 lower hyperbolic prediction bound of Y , in correspondence of $N = 2 \cdot 10^6$ cycles; instead according to IIW recommendations the characteristic S-N curve is obtained by translating the median S-N curve at .05 quantile of the $Y(x)$ Normal distribution, by using the lower bound of the 75% confidence interval of $\mu_Y(x)$ and the upper bound of the 75% confidence interval of σ_Y .

If we compare characteristic values $Y_{c,IIW}$ and $Y_{c,EN3}$ at a reference log-stress range, x^* , we have:

- $Y_{c,IIW}$ is the .05 quantile of the Normal distribution $\tilde{Y}(x^*)$ with parameters $\mu_{Y,l}$ and $\sigma_{Y,u}$, where $\mu_{Y,l}$ is the lower bound of the 75% confidence interval for μ_Y (that is the interval which in repeated sampling will contain the actual value of μ_Y with probability 75%) and $\sigma_{Y,u}$ is the upper bound of the 75% confidence interval

for σ_Y (that is the interval which in repeated sampling will contain the actual value of σ_Y with probability equal to 75%);

- $Y_{c,EN3}$ is the lower bound of the 95% prediction interval for $Y(x^*)$; it represents the lower bound of the interval that will contain the value $Y(x^*)$ with probability equal to 95%, if the experiment at $x = x^*$ is run at some time in the future.

Following numerical example shows the difference between the IIW-based statistical approach and the Eurocode-based statistical approach in estimation of the characteristic strength at $2 \cdot 10^6$ cycles.

Tab. 1 Numerical table

S [MPa]	N [MPa]	$x = \log(S)$	$x = \log(S)$	$x \cdot y$	x^2	y^2	$(x - \bar{x})^2$
108	1077000	2.0334	6.0322	12.2661	4.1348	36.288	0.0486
108	800000	2.0334	5.9031	12.0035	4.1348	34.8465	0.0486
139	597000	2.1430	5.7760	12.3780	4.5925	33.3619	0.0123
139	537000	2.1430	5.7300	12.2794	4.5925	32.8326	0.0123
202	204000	2.3054	5.3096	12.2406	5.3146	28.1922	0.0027
202	188000	2.3054	5.2742	12.1588	5.3146	27.8167	0.0027
202	107000	2.3054	5.0294	11.5945	5.3146	25.2947	0.0027
265	79000	2.4232	4.8976	11.8682	5.8721	23.9868	0.0287
265	70000	2.4232	4.8451	11.7409	5.8721	23.4750	0.0287
265	42000	2.4232	4.6231	11.2033	5.8721	21.3744	0.0287
Sum →		22.5387	53.4204	119.7331	51.0149	287.5684	0.2158

II.6.1 Numerical example

Fatigue test results from the numerical example in Section 4.3 of Commentary to Eurocode 3, part 1-9 (p.122 in [74]) are considered; the experimental stress ranges, S , and number of cycles to failure, N , are presented in the first two columns of Tab. 1.

The S-N curve parameters and the sums of square deviations are computed by using numerical results presented in Tab. 1:

$$\hat{m}_1 = \frac{n \cdot \sum_{i=1}^n (x_i \cdot y_i) - \sum_{i=1}^n x_i \cdot \sum_{i=1}^n y_i}{n \cdot \sum_{i=1}^n (x_i)^2 - \left(\sum_{i=1}^n x_i \right)^2} = \frac{10 \cdot 119.7331 - 22.5387 \cdot 53.4204}{10 \cdot 51.0149 - 22.5387^2} = -3.102$$

$$\hat{m}_0 = \bar{y} - \hat{m}_1 \bar{x} = 12.334$$

$$S_{xx} = \sum_{i=1}^n x_i^2 - \frac{\left(\sum_{i=1}^n x_i \right)^2}{n} = 51.0149 - \frac{22.5387^2}{10} = 0.216$$

$$S_{yy} = \sum_{i=1}^n y_i^2 - \frac{\left(\sum_{i=1}^n y_i\right)^2}{n} = 287.5684 - \frac{53.4204^2}{10} = 2.194$$

$$S_{xy} = \sum_{i=1}^n x_i \cdot y_i - \frac{\sum_{i=1}^n y_i \cdot \sum_{i=1}^n x_i}{n} = 119.7331 - \frac{22.5387 \cdot 53.4204}{10} = -0.669$$

where $n=10$ is the sample size.

The logarithm of the stress range at $2 \cdot 10^6$ cycles, on the mean regression line, is:

$$x_{cLS} = \frac{\log(2 \cdot 10^6) - \hat{m}_0}{\hat{m}_1} = \frac{\log(2 \cdot 10^6) - 12.334}{-3.102} = 1.945$$

The sample standard deviation is:

$$\hat{\sigma} = \sqrt{\frac{S_{yy} - \hat{m}_1 \cdot S_{xy}}{n - 2}} = \sqrt{\frac{2.194 + 3.102 \cdot (-0.669)}{8}} = 0.1214$$

IIW approach

By using $t_{.125,8} = -1.240$ and $\chi_{.125,8}^2 = 3.797$ in Equation 19, the characteristic S-N curve for the considered data-set is obtained:

$$y = \hat{m}_0 + \left(\frac{t_{.125,n-2}}{\sqrt{n}} - 1.645 \sqrt{\frac{8}{\chi_{.125,8}^2}} \right) \cdot \hat{\sigma} + \hat{m}_1 x = 12.334 - 0.338 - 3.102 \cdot x$$

The characteristic value of the logarithm of the stress range at $2 \cdot 10^6$ cycles is:

$$x_c = \frac{\log(2 \cdot 10^6) - 12.334 + 0.338}{-3.102} = 1.836$$

It follows:

$$S_c = 10^{x_c} = 68.53 \text{ MPa}$$

Eurocode approach

By using $t_{.05,8} = -1.860$ in Equation 22, the characteristic S-N curve for the considered data-set is obtained:

$$y = \hat{m}_0 + t_{.05,n-2} \cdot \hat{\sigma} \cdot \sqrt{1 + \frac{1}{n} + \frac{(x_{cLS} - \bar{x})^2}{\sum_{i=1}^n (x_i - \bar{x})^2}} + \hat{m}_1 x = 12.334 - 0.280 - 3.102 \cdot x$$

The characteristic value of the logarithm of the stress range at $2 \cdot 10^6$ cycles is:

$$x_c = \frac{\log(2 \cdot 10^6) - 12.334 + 0.280}{-3.102} = 1.854$$

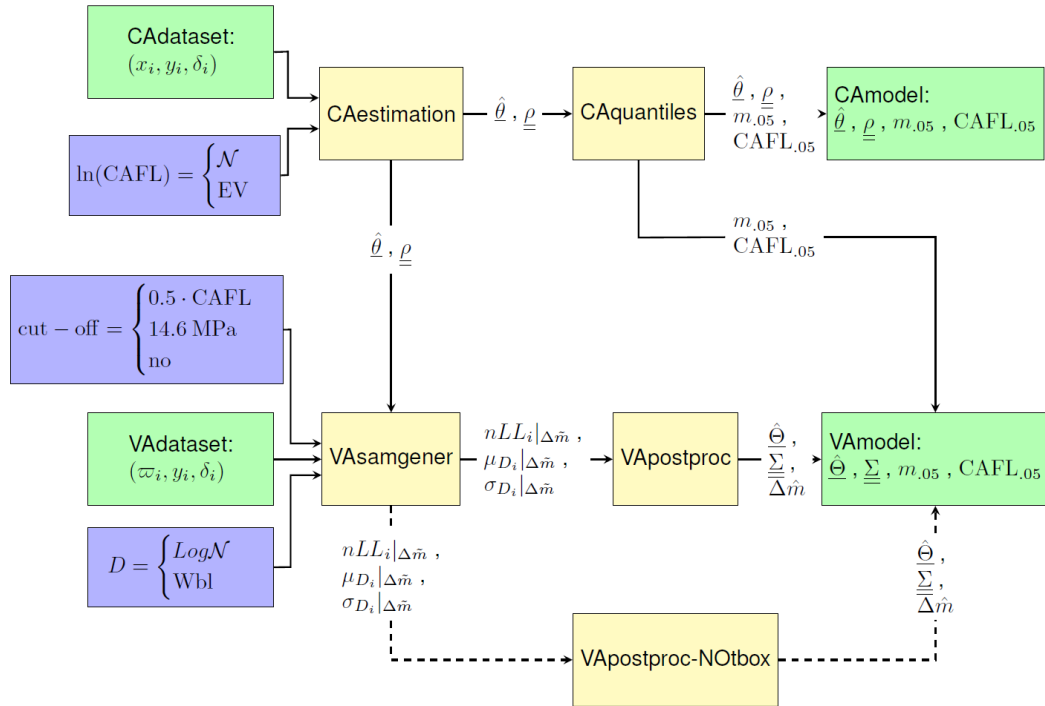
It follows:

$$S_c = 10^{x_c} = 71.50 \text{ MPa}$$

In this case Eurocode approach gives slightly higher estimate of the characteristic strength at $2 \cdot 10^6$ cycles, with respect to IIW approach (+ 4.4%).

III Matlab Toolboxes

III.1 Toolbox TB1



Legend:

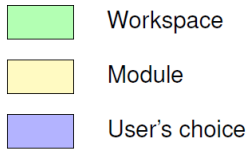


Fig. 3 Toolbox TB1.

This Toolbox is available for download at: <https://infoscience.epfl.ch/record/213809?ln=en>

III.2 Toolbox TB2

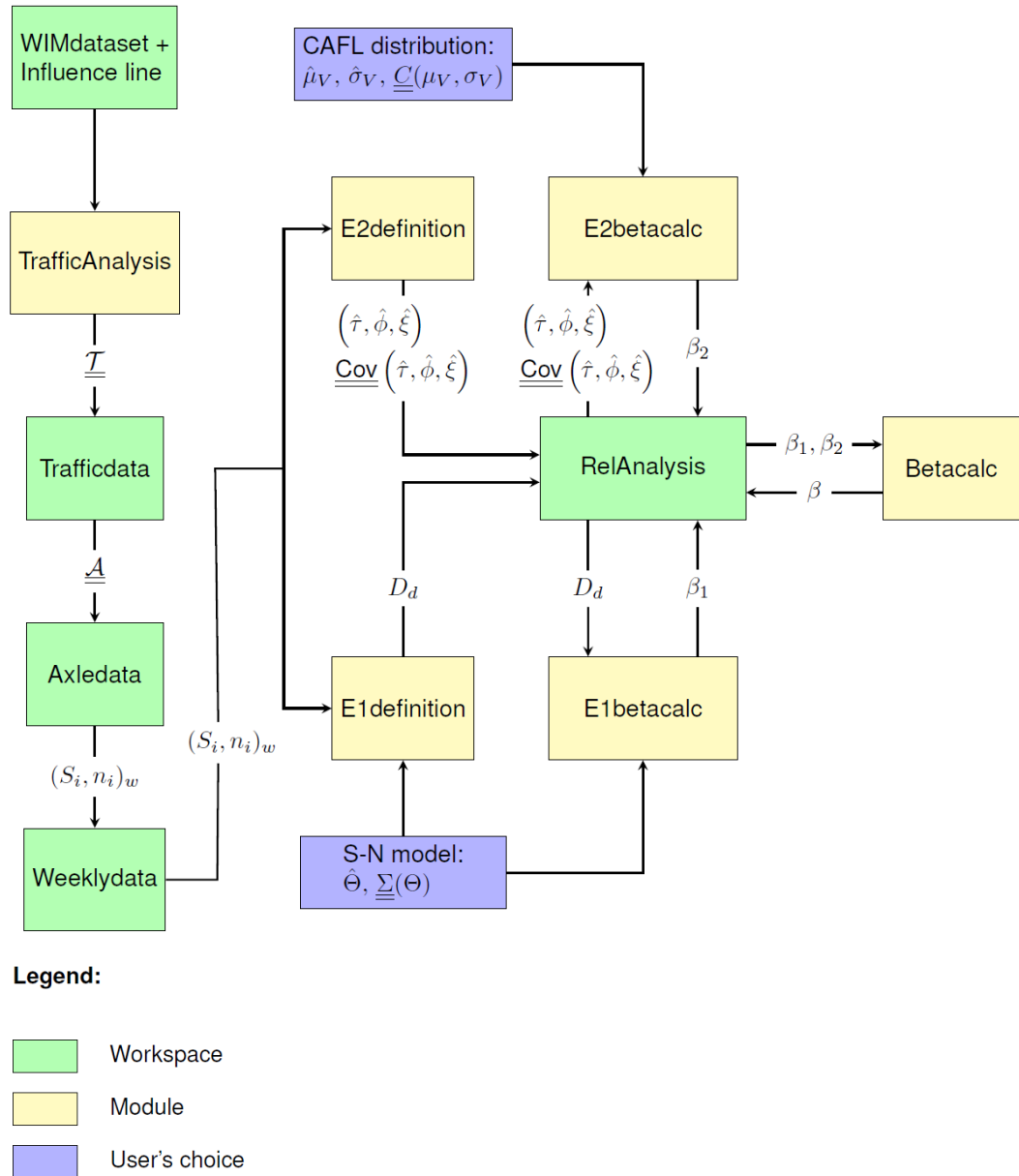
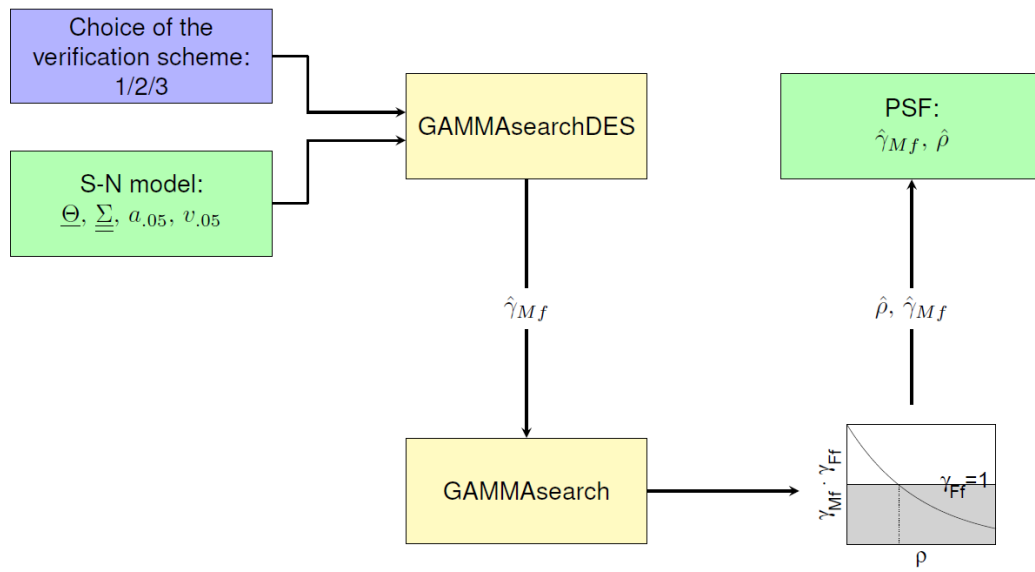


Fig. 4 Toolbox TB1.

This Toolbox is available for download at: <https://infoscience.epfl.ch/record/213809?ln=en>

III.3 Toolbox TB3



Legend:

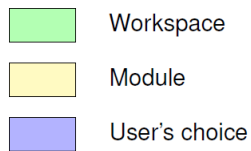


Fig. 5 Toolbox TB1.

This Toolbox is available for download at: <https://infoscience.epfl.ch/record/213809?ln=en>

Glossary

Terme	Signification
AASTHO	American Association of State Highway and Transportation Officials (AASTHO)
AGB	Groupe de travail recherche en matière de ponts (AGB)
AIMSUN	Advanced Interactive Microscopic Simulator for Urban and Non-Urban Networks, TSS (AIMSUN)
CA	Constant amplitude (CA)
CAFL	Constant amplitude fatigue limit (CAFL)
CDF	Cumulative distribution function (CDF)
ECSS	European Cooperation for Space Standardization (ECSS)
EN	European Standard (EN)
EPFL	Ecole Polytechnique Fédérale de Lausanne (EPFL)
EV	Extreme Value (EV)
FAT	Fatigue classes (FAT)
FE	Finite element (FE)
FL	Finite ligfe (FL)
FLM	Fatigue Laod Model (FLM)
FORM	First order reliability method (FORM)
GEV	Generalized extreme value (GEV)
GTW	Gross total weight (GTW)
HCF	High cycle fatigue (HCF)
ICOM	Laboratoire de la construction métallique, EPFL (ICOM)
IIW	International Institute of Welding (IIW)
JCSS	Joint Committee Structural Safety (JCSS)
MCS	Monte-Carlo Simulations (MCS)
ML	Maximum Likelihood (ML)
MPa	Megapascal (MPa)
LAVOC	Laboratoire des voies de circulation, EPFL (LAVOC)
LEFM	Linear elastic fracture mechanics (LEFM)
LS	Least-squares (LS)
PDF	Probability density function (PDF)
PHP	Personal Home Page, Hypertext Preprocessor (PHP)
PLR	Profile likelihood ration (PLR)
PMC	Probabilistic Model Code (PMC)
RFL	Random fatigue limit (RFL)
RMC	Root-mean-cube (RMC)
SARTC CSACR	Swiss automatic road traffic counts (SARTC) <i>Comptage suisse automatique de la circulation routière (CSACR)</i>
SIA	Société suisse des Ingénieurs et Architectes (SIA)
SORM	Second order reliability method (SORM)
SQL	Structured Query Language (SQL)

STRUREL	Structural Reliability Analysis Program-System (STRUREL)
TSS	Transport Simulation Systems (TSS)
VA	Variable amplitude (VA)
VAFL	Variable amplitude fatigue limit (VAFL)
VSS	Association suisse des professionnels de la route et des transports (VSS)
WIM	Weigh in Motion (WIM)

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Clôture du projet



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RECHERCHE DANS LE DOMAINE ROUTIER DU DETEC

Version du 09.10.2013

Formulaire N° 3 : Clôture du projet

établi / modifié le : 12 Octobre 2016

Données de base

Projet N° : AGB 2010/003

Titre du projet : Simulations de trafic intégrant la détermination d'indices de performance structurale

Echéance effective : 31 Octobre 2016

Textes :

Résumé des résultats du projet :

Les résultats de la recherche sont séparés en deux parties, qui se rejoignent dans une méthodologie complète permettant l'évaluation structurale des ouvrages d'art et des chaussées : le trafic et l'effet de ce dernier sur les chaussées et dans l'évaluation structurale des ponts routiers sous l'effet du trafic, en particulier vis-à-vis de la fatigue.

En ce qui concerne les charge de trafic poids lourds, l'étude s'est basée sur des investigations statistiques des comptages routiers et des stations de pesage en marche (WIM), sur des études vidéo et d'accidentologie, à différentes échelles de temps (de jour à années). Elle a permis de déterminer un modèle de comportement des poids lourds. Grâce à ce modèle, qui doit être adapté à chaque situation spécifique, il est alors possible d'évaluer de manière précise les effets du trafic total (véhicules légers et poids lourds) passé, actuel et futur, en effectuant des simulations de trafic sur la durée souhaitée (en général une semaine). Afin de démontrer la démarche, cette dernière a été effectuée pour trois cas d'étude :

- Denges, autoroute A1, proche du pont sur la Venoge sur l'axe Lausanne - Genève (le réseau complet avec les nœuds a été développé : Mex - Maladière - Ecublens - Morges - Genève Aéroport)
- Monte Ceneri, Autoroute A2, sur l'axe Lugano - Bellinzona
- Aarwangen, pont sur l'Aare, route cantonale n°244 Langenthal - Niederbipp-Oensingen

Les comparaisons entre simulations et trafic réels démontrent l'excellente qualité des simulations en termes de débits, de vitesses moyennes et de variations sur 10 répliques.

Ce projet a permis la création de passerelles entre différents outils et ainsi l'utilisation des résultats des simulations pour l'évaluation des ouvrages d'art et des dégradations des chaussées soumis au trafic routier, via des indices de performance, v. Dans le cas de la fatigue, ceci s'effectue à l'aide de passerelles développées entre AIMSUN (logiciel de simulation de trafic) et les routines Matlab de détermination des efforts dans les sections/détails déterminants.

Pour estimer les courbes de résistance à la fatigue nécessaire au calcul de l'indice de performance à la fatigue, l'approche combinée « maximum likelihood » (ML) et Simulations de Monte-Carlo (MCS) proposée constitue un outil puissant pour la redéfinition de courbes S-N normalisées (caractéristiques) et de la partie à amplitude variable (seconde pente) de ces mêmes courbes, ainsi qu'une évaluation plus réaliste de la valeur critique du cumul de dommage. Il est important d'insister sur le fait que l'efficacité de l'approche proposée et de la confiance dans la vérification en fatigue sont conditionnées par les résultats à disposition contenant suffisamment d'information (sous CA et VA) dans la région à grand nombre de cycles ($N > 5$ millions de cycles), ce qui fait souvent défaut à ce jour.

Une nouvelle méthodologie développée dans cette étude avec les outils ML-MCS permet en outre d'effectuer des estimations de la fiabilité en fatigue des ponts existants en se servant de trafics simulés. Dans cette méthodologie, la probabilité de rupture en fatigue est définie en prenant en considérant la probabilité conjointe de l'événement de dépassement de la CAFL et de l'événement d'accumulation de dommage critique. L'estimation de la fiabilité en fatigue a été appliquée au cas d'étude du pont existant de la Venoge. Bien que le pont possède un indice de fiabilité élevé, elle démontre l'inexactitude de la vérification avec l'Eurocode pour estimer l'évolution de l'indice de fiabilité durant les 100 ans de durée de vie du pont. Même lorsqu'ils sont appliqués à des assemblages soudés, les outils développés ont une applicabilité générale.



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Atteinte des objectifs :

Les objectifs de la recherche ont été en grande partie atteints :

- Détermination d'un modèle de comportement des poids lourds : les résultats établis et les démarches proposées permettent d'atteindre, en fonction du niveau de détail souhaité et adapté à chaque cas étudié, un modèle de trafic routier.
- Démarche pour atteindre, en fonction du niveau de détail souhaité et adapté à chaque cas étudié, un modèle de trafic routier : la démonstration est faite, au travers de trois cas d'étude, de la manière d'effectuer des simulations de trafic précises pour les effets du trafic total (véhicules légers et poids lourds) passé, actuel et futur.
- Création de passerelles entre les différents outils et, ainsi, possibilité d'utilisation des résultats des simulations de trafic pour l'évaluation des ouvrages d'art et des dégradations des chaussées soumis au trafic routier, via des indices de performance.
- Définition des indices de performance structurale pour un ouvrage d'art et pour une chaussée bitumineuse; de plus amples développements sont cependant souhaitables pour la détermination de ces indices.
- Développement de l'approche combinée « maximum likelihood » (ML) et Simulations de Monte-Carlo (MCS) en un outil puissant pour la redéfinition de courbes de fatigue normalisées sous amplitude constante et variable.
- Nouvelle méthodologie, avec les outils ML-MCS, permettant d'effectuer des estimations de la fiabilité en fatigue des ponts existants.

Déductions et recommandations :

- Pour définir un modèle de trafic routier, cette recherche met en évidence la nécessité de mener des études locales plutôt que de se baser sur un modèle appliqué à l'échelle nationale en termes de composition et de charges de trafic.
- L'approche proposée pour estimer les courbes de fatigue sous amplitude variable (et a fortiori les courbes sous amplitude constante) peut être utilisée pour améliorer les normes à la fois pour les nouvelles structures et pour les structures existantes. Le fait est que le besoin d'avoir des méthodes plus précises est important pour les structures existantes, où l'évaluation fournit des valeurs de durées de vie restantes. Par conséquent, la redéfinition de courbes de fatigue dans les normes d'évaluation de ponts existants doit être discutée ; les courbes actuelles de l'Eurocode pour les nouvelles structures peuvent être conservées pour le moment.
- En fatigue, les valeurs de l'indice de fiabilité obtenues sont probablement trop élevées, le coefficient de variabilité pour les effets du trafic étant très faible de par la répétition de simulations de courtes durées, une semaine (théorème de limite centrale). Il est recommandé d'effectuer des simulations de plus longues durées.
- Un manque de résultats d'essais (sous CA et VA) dans la région à grand nombre de cycles ($N > 5$ millions de cycles) a été identifié. Pour permettre une meilleure redéfinition des courbes de fatigue et l'utilisation des méthodes avancées développées, il faut investir dans de nouveaux programmes d'essais de fatigue des détails les plus utilisés.

Publications :

D'Angelo, L., Nussbaumer, A., Féart, M.-A., Dumont, A.-G. (2013), Fatigue life assessment of existing motorway bridge, 5th Int. Conf. on Structural Eng., Mechanics and Computation, Cape Town, South Africa, Sept. 2-5.
D'Angelo, L., Rocha, M., Nussbaumer, A., Brühwiler, E. (2014), SNP Fatigue Curves using Maximum Likelihood, paper 461, EUROSTEEL 2014, Sept. 10-12, Naples, Italy.
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D'Angelo, L., Faber, M. H., Nussbaumer, A. (2015), Calibration of partial safety factors for fatigue design of steel bridges, paper 621, ICASP12, Vancouver, Canada, July 12-15.
D'Angelo, L. (2015), Probabilistic approach for fatigue evaluation of welded connections with application to road steel bridges, PhD thesis EPFL no 6827, EPF Lausanne.

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Formulaire N° 3 : Clôture du projet

Appréciation de la commission de suivi :

Evaluation :

Les démarches conduites et les résultats obtenus sont évalués de manière positive par la commission, qui relève également que les objectifs principaux de la recherche ont été atteints.

Au chapitre des conclusions de la recherche, la commission souligne ses apports importants, notamment en matière

- de méthodologie de traitement des données de trafic CSACR et WIM,
- d'élaboration de simulations de trafic,
- d'établissement, à partir d'un modèle de leur comportement, d'un modèle de circulation des poids lourds,
- de manière (démontrée à l'aide de trois cas d'étude) d'effectuer des simulations précises de trafic pour les effets du trafic passé, actuel et futur,
- de définition du trafic sollicitant réellement un ouvrage et, en conséquence, d'estimation des sollicitations futures afférentes,
- de définition des indices de performance structurale pour les ouvrages d'art et pour les chaussées revêtues de bitume,
- d'élaboration d'un outil pour la redéfinition de courbes de fatigue normalisées sous amplitude constante et variable,
- de développement d'une nouvelle méthodologie pour estimer la fiabilité en fatigue des ponts existants ou encore la mise en évidence d'une ambiguïté législative en ce qui concerne la vitesse des poids lourds.

Mise en oeuvre :

La recherche fournit des conclusions très utiles pour le dimensionnement et/ou la vérification des infrastructures routières (ponts, chaussée) en ce qui concerne la définition respectivement le pronostic de charges et de composition du trafic ayant sollicité, sollicitant ou étant appelé à solliciter ces infrastructures, en particulier

- la nécessité, en termes de composition et de charges de trafic, de mener des études locales plutôt que de se baser sur un modèle appliqué à l'échelle nationale;
- la prise en compte du comportement effectif des poids lourds.

Les résultats de la recherche permettront d'actualiser les facteurs d'équivalence des poids lourds de la norme en vigueur.

La meilleure précision de la détermination du trafic pondéral pourra servir de base au niveau de l'approche du dimensionnement et du renforcement des chaussées.

En matière de comportement à la fatigue des ouvrages, tant au niveau de leur dimensionnement que de leur vérification, la recherche conduit à proposer l'utilisation d'une approche d'estimation des courbes de fatigue sous amplitude variable et sous amplitude constante. Ayant mis en évidence le fait que les valeurs de l'indice de fiabilité obtenues sont probablement trop élevées, au motif de la répétition de simulations de courbes durées, la recherche recommande d'effectuer des simulations de plus longues durées.

L'ambiguïté relevée au niveau des dispositions législatives en vigueur en ce qui la vitesse des poids lourds pourrait devoir faire l'objet de révisions légales, en intégrant l'aspect de la sécurité routière dans le contexte de cette réflexion.

Besoin supplémentaire en matière de recherche :

L'identification d'un manque de résultats d'essais (sous CA et VA) dans la région à grand nombre de cycles ($N > 5$ millions de cycles) nécessite, afin de permettre une meilleure redéfinition des courbes de fatigue et l'utilisation des méthodes avancées développées, de mettre en oeuvre de nouveaux programmes d'essais de fatigue des détails les plus utilisés. Des études complémentaires sur la répartition du trafic pondéral permettraient de valider ou d'actualiser les hypothèses de la SN 640 320 à son article 17.

A son chapitre 10.3, le rapport de recherche met en outre en évidence des besoins en matière de recherche, notamment au niveau de données et de recensement de trafic, de développement d'outils complémentaires pour la simulation de trafic, d'étude de l'évolution de la dégradation des revêtements bitumineux sur les ponts ou encore d'amélioration de la concordance entre les résultats des systèmes de relevés/mesures de données routières.

Influence sur les normes :

Une redéfinition des courbes de fatigue dans les normes d'évaluation des ponts existants doit être discutée ; les courbes actuelles de l'Eurocode pour les nouvelles structures peuvent être conservées pour le moment.

L'approche proposée pour estimer les courbes de fatigue sous amplitude variable (et a fortiori sous amplitude constante) peut être utilisée pour améliorer les normes à la fois pour les nouvelles structures et pour les structures existantes, pour lesquelles le besoin d'avoir des méthodes plus précises est important, l'évaluation fournissant des valeurs de durées de vie restantes.

Président/Présidente de la commission de suivi :

Nom : Putallaz

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Signature du président/ de la présidente de la commission de suivi :

Index des rapports de recherche en matière de route

L'index actuel des rapports publiés dernièrement peut être téléchargé sur le site www.astra.admin.ch (Services → Recherche en matière de routes → Downloads → Formulaires).