An approach for fatigue life assessment of a road bridge based on measured corrosion and actual traffic loading

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Abstract. Fatigue and corrosion play a significant role in the aging of steel bridges, especially in a marine environment. An absence of fatigue strength curves for details which are exposed to corrosive environments, realistic traffic load models and accurate damage accumulation models increase the susceptibility of fatigue damage of steel bridges. In this paper, remaining fatigue life of a road bridge is assessed based on the measured corrosion wastage and more realis-tic traffic loads. Corrosion fatigue is also studied as it is one of the main factors that reduce the fatigue life and recently proposed fatigue strength curve for structural details exposed to corrosive environments is utilized for this study. A non-linear fatigue damage accumulation model is used for life assessment apart to the conventional damage model, Miner's rule. The calculated fatigue lives are compared and discussed in quantitative manner to identify the most conservative remaining life and corresponding methods.

1. Introduction

Bridges are subjected to multiple recurring traffic loads that may significantly fall below their structural resistance limit. Structural damage accumulated continually over a period of time results in localized and cumulative failure processes known as fatigue. It is well known that all bridges are subjected to environment assisted cracking (EAC) which includes three major types such as stress-corrosion cacking (SCC), hydrogen damage (HD) and corrosion fatigue (CF) due to different environment, structural and metallurgical factors [1-7].

The steel bridges are subjected to different aggressive corrosive environment, and it causes the time dependent loss of protective coating and material loss due to corrosion. When the steel is exposed corrosive environment and alternative cyclic stresses, then steel is subjected to corrosion fatigue. It is obvious that corrosion fatigue is significant compared to the damage from fatigue and corrosion [1-4,6-8,11]. The corrosion affects the material loss, and it changes the cross-sectional properties of the members due to changes of the surface roughness, irregularities, corrosion pits. The deterioration process caused by EAC will affect the integrity of the bridges by the degradation of material strengths. There may be a reduction in remaining fatigue life of the steel bridges due to changes in structural behavior and stiffness [11]. Fatigue is still less understood regarding the cause of formation and failure mechanism with respect the structural details even though fatigue is considered as one of the most critical form of failures of steel structures. The aging of bridges needs the attention as the replacement of the bridges are not economically viable and fatigue pays a significant role in their service life.

Meanwhile much research shows that corrosion and fatigue are the main mechanism for deterioration. There are a quite lot of research work have been done to study the interaction effect of corrosion and fatigue. Creating improved technologies for capturing the fatigue phenomena and undertaking a reliable assessment of the fatigue damage are necessary for steel bridges [5,8-14].

Main objective of this this paper is to spot and assess the importance of using an accurate fatigue assessment procedure which consists of accurate models for stress evaluation, S-N curves, and fatigue damage accumulation. The applicability and significance of the recently proposed generalized formula of the S-N curve is confirmed by applying to a case study road bridge. Further, a conventional method predicted fatigue lives are compared with recently developed assessment procedures which are utilized to study remaining life of the bridges.

2. Recently proposed fatigue damage models, strength curves and frameworks

2.1. Fatigue strength curve for structural details exposed to corrosive environments

The fatigue performance of steel components of these structures is affected by the exposed environment. Determination of fatigue strength of structural joints and constructional elements in corrosive media or corrosive environment has been studied. The recently proposed fatigue damage model aimed the fatigue strength of structural joints and constructional elements in corrosive environments and modification has been done by considering the design fatigue strength curves of constructional details and detail classes [9-11].

The recently proposed formula for fatigue strength of a corroded detail is shown as follows,

If $\Delta\sigma_{cor} \geq \Delta\sigma_{D,cor}$,

$$\Delta\sigma_{cor} = \Delta\sigma_D \left[N_{f,LCF}^{c} N_{f,CAFL}^{1/m}\right] N_R^{(-c-1/m)} \; where \; c = \left. log\left[\frac{\Delta\sigma_D}{\Delta\sigma_{D,cor}}\right] \middle/ log\left[\frac{N_{f,CAFL}}{N_{f,LCF}}\right] \right. \quad (1)$$

If $\Delta\sigma_{cor} \leq \Delta\sigma_{D,cor}$

$$\Delta\sigma_{cor} = \Delta\sigma_{D,cor}\left[N_{f,CAFL}^{-\acute{c}}\right] N_R^{\acute{c}} \; where \; \acute{c} = \left. log\left[\frac{\Delta\sigma_{D,cor}}{\Delta\sigma_{L,cor}}\right] \middle/ log\left[\frac{N_{f,CAFL}}{N_{f,VAFL}}\right] \right. \quad (2)$$

where $\Delta\sigma_D$ is the stress range at the fatigue curve slope changing point, corresponding to the $N_{f,CAFL}$ cycles. The $-{}^{1}/_{m}$ is the slope of the fatigue strength curve uncorroded details, where m is equal to 3 when $\Delta\sigma \geq \Delta\sigma_D$, equal to 5 when $\Delta\sigma_D \geq \Delta\sigma > \Delta\sigma_L$ and infinite when $\Delta\sigma \leq \Delta\sigma_L$, where $\Delta\sigma_L$ is the fatigue endurance limit of the detail corresponding to $N_{f,VAFL}$. $\Delta\sigma_{D,cor}$ is the stress range for $N_{f,CAFL}$ cycles at the intersection of the two corroded fatigue curves. $N_{f,CAFL}$ is the number of cycles to fatigue failures of uncorroded details at the intersection point [11].

The corrosive environment dependent parameters have been determined conservatively for corrosive environments such as urban and maritime environments and tabulated in the Table 1 and Table 2 [9-11].

Table 1. Parameters used in proposed fatigue strength curve of corroded details

Parameter	Constructional details in Eurocode	Constructional details in DNV code
$N_{f,LCF}$	10^4	10^4
$N_{f,CAFL}$	$5x10^6$	10^7
$N_{f,VAFL}$	10^8	10^8
$\frac{\Delta\sigma_L}{\Delta\sigma_D}$	0.549	0.631

Table 2. Mean values and conservative values used in fatigue strength curve of corroded details

Corrosion parameters	Constructional details in Eurocode				Constructional details in DNV code			
	Mean value	Conservat -ive value	Mean value	Conservat -ive value	Mean value	Conservat ,ive value	Mean value	Conservat -ive value
$\frac{\Delta\sigma_{D,cor}}{\Delta\sigma_D}$	0.497	0.308	0.641	0.536	0.46	0.27	0.61	0.50
$\frac{\Delta\sigma_{L,cor}}{\Delta\sigma_L}$	0.356	0.175	0.518	0.40	0.356	0.175	0.518	0.40

2.2. Non-linear fatigue damage model based only on S-N curve parameters

A recently proposed non-linear fatigue damage model, which take loading sequence effect more accurately than Miner's rule, is adapted in this paper to calculate cumulative fatigue damage by utilizing nominal and/or hot-spot approach [12]. The model is applicable for several engineering fields. The equation illustrates an effective and simple fatigue damage model which has been developed based on fatigue damage evolution curve. It gives a damage at a particular level.

$$D_i = 1 - \left[1 - \frac{n_i}{N_i}\right]^{\delta_i} = 1 - \left[1 - \frac{n_{(i+1),eff}}{N_{i+1}}\right]^{\frac{\delta_{i+1}}{\mu_{i+1}}} \tag{3}$$

where D_i is the damage at load level i when exposed to a given stress amplitude (or range) σ_i for n_i cycles. $n_{(i+1),eff}$ is the effective number of cycles corresponding to σ_{n+i} at level $i+1$. N_i and N_{i+1} are the cycle to failure number of cycles which can be taken from *S-N* curves given in design standards and regulations. The μ_i and δ_i are model parameters that depend on *N* and stress levels.

$$\delta_i = \frac{-1.25}{lnN_i} \tag{4}$$

$$\mu_i = \left(\frac{\sigma_{i-1}}{\sigma_i}\right)^2 \tag{5}$$

2.3. Life assessment framework/guidelines

The life assessment framework is recently proposed to be applied for existing steel bridges [7-11]. The newly introduced formula of S-N curve for corroded details and Miner's rule were used to find the remaining fatigue life which exposed to corrosive environment or EAC. In this study, previously proposed framework was modified by adapting recently proposed non-linear damage model mentioned in section 2.2 in-stead of Miner's rule. There are five steps of the framework. First step is to do the structural analysis to replicate the current state of the bridge. Critical element is found by using structural analysis and it's necessary to determine the stress spectrum of the identified critical element. The remaining fatigue life is calculated by using conventional approach and newly proposed method. [7-11]. The current state of the structural details is categorized as corroded or uncorroded or /and in a corrosive environment.

3. Case study bridge: Description and its current state

The considered bridge was constructed in 1937 which is in Strand Municipality, Norway. The view of the bridge is shown in Figure 1. The bridge was partially destroyed by floods and rehabilitated and rebuilt in 1942 with modifications as per today. There are indications that bridge has exposed to corrosive environment and subjected to increased load cycles.

Figure 1. Considered road bridge

3.1 Geometrical information and material properties

The considered bridge has two simple spans of non-composite sections, each with an equal length of 19.5m and an end span of concrete T- beams with 12.7m in length as shown in Figure 2. There are two lane single carriageway and superstructure is supported by two pillars. The non-composite section consists of a reinforced concrete deck. The main steel girder consists of three evenly spaced rolled sections, designated as DIP 95 as shown in Figure 3. The total width of concrete deck has a total width of 5.82m with an average depth of 190mm as shown in the Figure 3.

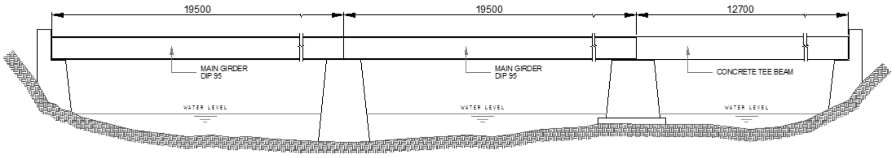

Figure 2. Longitudinal section of bridge

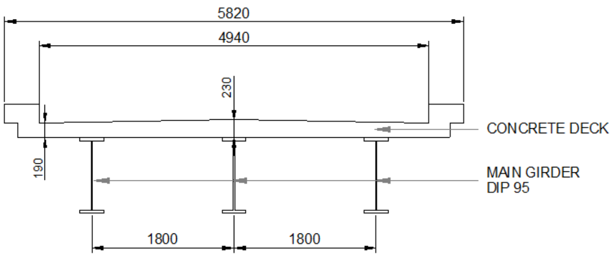

Figure 3. Cross sectional view and main girder steel beams (DIP 95)

The DIP 95 is doubly symmetric I shape plate girder beam with depth, flange width, web thickness and flange thickness 950mm, 300mm, 19mm and 36mm respectively. The cross-sectional properties such as gross area, 2nd moment of area about major axis and elastic section modulus about major axis are 390.55 cm^2, 572953 cm^4 and 12062 cm4 respectively. The grade of the steel is S275, where characteristic yield strength and modulus of elasticity 275 MPa and 200GPa respectively.

3.2 Degradation, damage and defects description

The bridge inspection reports, and current visual inspection results provides of coating loss and corrosion in the bridge girders as shown in Figure 4. It is reported that the bridge is exposed to uniform/patch corrosion and those are in the mid-span of the exterior girder and bottom surface of the top flange. In general, it is found that bottom of the top flange, bottom and top surface of the bottom flange have exposed to corrosion as shown in Figure 5. Visible cracks were not identified during the visual inspection of any steel portion of the bridge. A maximum of 4mm uniform corrosion is recorded in the midspan of the exterior girder. The thickness reduction of the uniform corrosion changes the structural stiffness by changing cross sectional properties. Therefore, degraded DIP 95 cross section at mid span is shown in Figure 5.

Figure 4. Bridge girders with corrosion

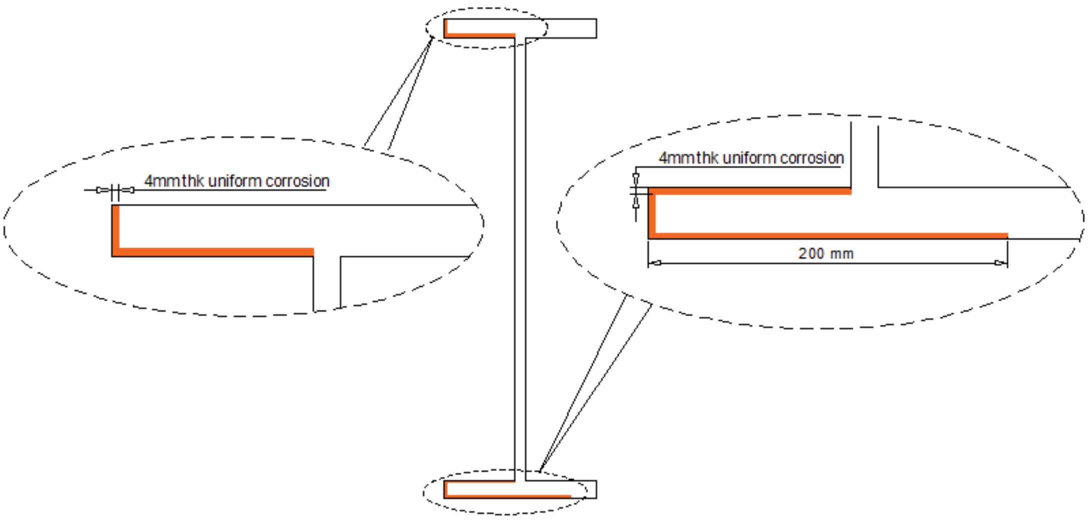

Figure 5. Corroded DIP 95 cross section at midspan of exterior girder with about 4mm thickness reduction at the top and the bottom surfaces.

4. Structural analysis of the bridge

4.1 Load models

Eurocode defines five different fatigue load models [13]. Fatigue load model 4 (FLM 4) is used to calculate the total fatigue damage accumulated through the design service life of the bridge as recommended for road bridges [13]. The total number of lorries crossing the bridge per year is 125 000 which was taken from Euro-code based on traffic category 3 which is main roads with low flow rates of lorries. The axle spacing, equal axle loads and percentages of heavy traffic for five lorries were taken from the Table 4.7 of Eurocode by considering medium distance road category [14].

In addition to Eurocode FLM 4, an alternative fatigue load model (AFLM) pro-posed for bridges in Norway [15] is used for fatigue life assessment. The AFLM consists of 5 lorries in FLM4 and additional six other light vehicles. The load and axle spacings of additional vehicles are shown in Table 3.

Table 3. Additional vehicle details of AFLM

Vehicle type	Axle spacing	Axle load
Combi	2.5	12
		12
Sedan	2.9	16
		16
Station	3.0	19
		19
SUV/Minivan	3.1	21
		21
Pickup/Van	3.2	29
		29
Tractor/Smaller trucks	3.5	64
		64

Probabilities for the six remaining vehicle types have been derived from the distribution of registered cars in Norway. Based on the annual average density of traffic (AADT) and the route's location, public road administration given traffic data estimates that 5% to 20% of all vehicles are heavy vehicles. FLM4 has an 8.56% share of heavy vehicles based on the expected AADT and the predicted number of trucks. FLM4 Between 8 and 15% of the five FLM4 cars on the bridge is likely to be ac-counted for by the AADT by categorizing to three scenarios, such as Scenario 1: AFLM with 4% heavy vehicles, Scenario 2: AFLM with 8.56% heavy vehicles, and Scenario 3: AFLM with 15% heavy vehicles. Probability of occurrence of different scenarios are shown in Table 4. The total number of vehicles crossing the bridge per year is 365 000.

Table 4. Probability of occurrence for the scenarios AFLM (1) One lane loaded (2) Both lanes loaded

Vehicle type	Traffic percentage of AFLM (%)		
	Scenario 1	Scenario 3	Scenario 3
Combi (1)	10.56	10.0584	9.35
Sedan (1)	10.56	10.0584	9.35
Station wagon (1)	9.12	8.6868	8.075
SUV/Minivan (1)	7.92	7.5438	7.0125
Pickup/Van (1)	6.00	5.715	5.3125
Tractor/Smaller trucks (1)	3.84	3.6576	3.40
Lorry 1 FLM4 (1)	0.80	1.712	3.00
Lorry 2 FLM4 (1)	0.20	0.428	0.75
Lorry 3 FLM4 (1)	0.60	1.284	2.25
Lorry 4 FLM4 (1)	0.10	0.214	0.375
Combi (2)	10.56	10.0584	9.35
Sedan (2)	10.56	10.0584	9.35
Station wagon (2)	9.12	8.6868	8.075
SUV/Minivan (2)	7.92	7.5438	7.0125
Pickup/Van (2)	6.00	5.715	5.3125
Tractor/Smaller trucks (2)	3.84	3.6576	3.40
Lorry 1 FLM4 (2)	0.80	1.712	3.00
Lorry 2 FLM4 (2)	0.20	0.428	0.75
Lorry 3 FLM4 (2)	0.60	1.284	2.25
Lorry 4 FLM4 (2)	0.10	0.214	0.375

4.2 Stress evaluation

The current bridge has two spans, and a single-span bridge was modelled. The bridge's behavior during its service life was done by the structural behavior under moving load. The influence line analysis was done for each single passage of vehicle and hence bending moments are established based on the influence line during the passage of each lorry, which will result in a stress history at the mid-span where severe corrosion is reported. Fatigue assessment for the current bridge classified as a safe life method adopting the recommended value for partial factor for fatigue (γ_{mf}) is 1.35. A 3m with of one notional lane was considered and therefor width of remaining area is 1.94m. For assessing the local effect with notional lanes, the single lorry is placed on the span cantered in the notional lane. One lane loading is analysed and obtained load distribution factor to exterior and interior girders are 0.55 and 0.14 respectively. The calculation of the nominal stress ranges of exterior girder involves multiplying the highest response by the distribution factor and the dynamic amplification factor of 1[12,16]. The result is then divided by the related section modulus of DIP 95 (corroded/uncorroded condition). The obtained stress ranges are shown in Table 5.

Table 5. Nominal stress ranges

Vehicle Type	ΔM_i (kNm)	Uncorroded state		Corroded state	
		$\Delta\sigma_i$ (MPa)	$\Delta\sigma_R$ (MPa)	$\Delta\sigma_i$ (MPa)	$\Delta\sigma_R$ (MPa)
Combi (1)	56.10	6.05	8.16	6.60	8.91
Sedan (1)	73.04	7.87	10.63	8.60	11.60
Station wagon (1)	73.04	7.87	10.63	8.60	11.60
SUV/Minivan (1)	94.60	10.20	13.76	11.13	15.03
Pickup/Van (1)	130.00	14.01	18.91	15.30	20.65
Tractor/Smaller trucks (1)	281.60	30.35	40.97	33.14	44.74
Lorry 1 FLM4 (1)	453.20	48.84	65.94	53.34	72.01
Lorry 2 FLM4 (1)	707.85	76.29	102.99	83.31	112.47
Lorry 3 FLM4 (1)	856.35	92.29	124.60	100.79	136.06
Lorry 4 FLM4 (1)	657.25	70.84	95.63	77.35	104.43
Lorry 5 FLM4 (1)	694.65	74.87	101.07	81.75	110.37
Combi (2)	56.10	6.05	8.16	6.60	8.91
Sedan (2)	73.04	7.87	10.63	8.60	11.60
Station wagon (2)	73.04	7.87	10.63	8.60	11.60
SUV/Minivan (2)	94.60	10.20	13.76	11.13	15.03
Pickup/Van (2)	130.00	14.01	18.91	15.30	20.65
Tractor/Smaller trucks (2)	281.60	30.35	40.97	33.14	44.74
Lorry 1 FLM4 (2)	453.20	48.84	65.94	53.34	72.01
Lorry 2 FLM4 (2)	707.85	76.29	102.99	83.31	112.47
Lorry 3 FLM4 (2)	856.35	92.29	124.60	100.79	136.06
Lorry 4 FLM4 (2)	657.25	70.84	95.63	77.35	104.43
Lorry 5 FLM4 (1)	694.65	74.87	101.07	81.75	110.37

5. Fatigue life assessment

5.1 Detail categories and fatigue strength curves

The main girder cross section is DIP 95 and it is a rolled I section. The main girder is subjected bending and therefore the bottom flange subjected to tensile stresses. If the fatigue stresses of the bottom flange are due to in-plane bending, the detail category 160 with constructional detail number 2 can be chosen according to Eurocode 3 [17]. The corresponding detail category-based S-N curve is shown Figure 6. This S-N curve is used for fatigue life assessment of uncorroded details of the girder.

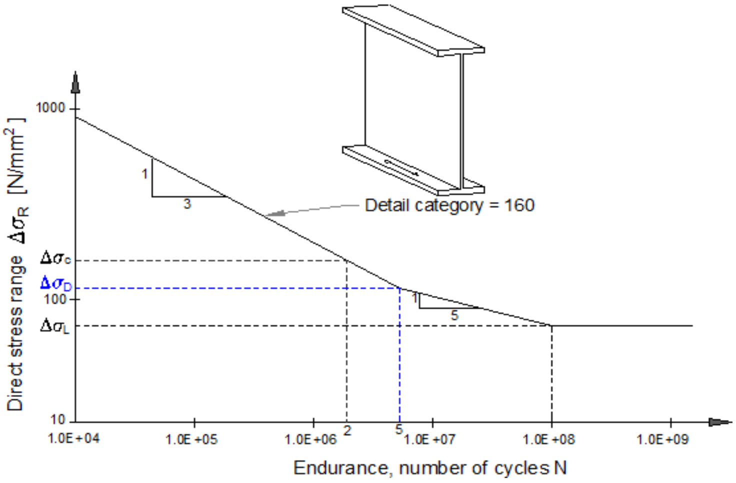

Figure 6. Design S-N curve for detail category 160 which is used for uncorroded DIP 95 girder

Fatigue strength curve for structural details exposed to corrosive environments, which is corroded DIP 95 girder, was obtained from the recently proposed formula [9-11] which formulation was discussed in the section 2.1. The S-N curve used for corroded details are shown in Figure 7, where $\Delta\sigma_D = 0.737 \times 160$ MPa.

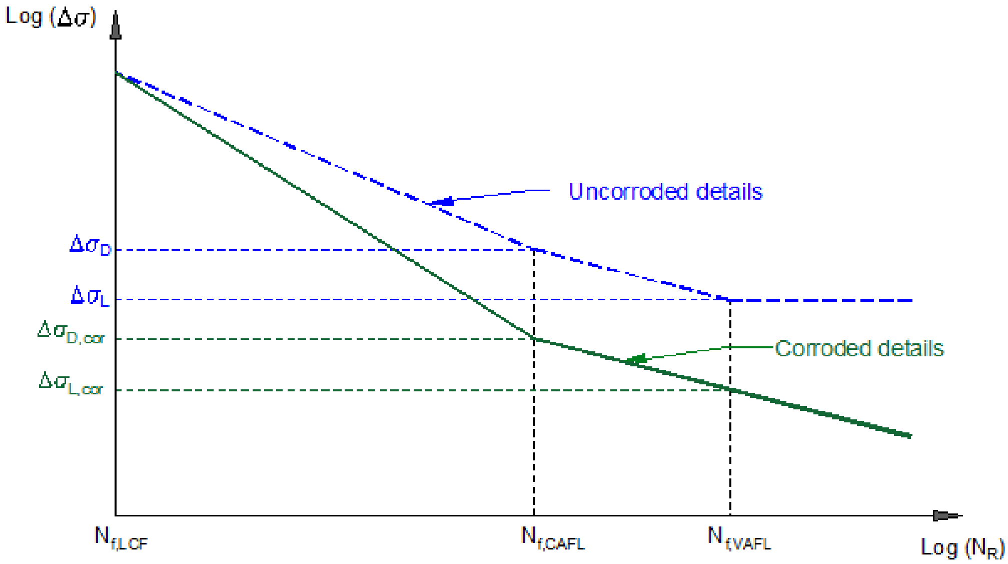

Figure 7. S-N curve used for corroded DIP 95 girder

5.2 Fatigue life calculation

Fatigue life of the critical location of the girder was calculated by four different methods [19-23]. Method 1: Life assessment using stress ranges due to FLM4, both corroded and uncorroded S-N curves and Miner's rule. Method 2: Life assessment using stress ranges due to AFLM, both corroded and uncorroded S-N curves and Miner's rule. Method 3: Life assessment using stress ranges due to FLM4, both corroded and uncorroded S-N curves and non-linear damage model described in section 2.2 [20-23]. Method 4: Life assessment using stress ranges due to AFLM, both corroded and uncorroded S-N curves and non-linear damage model described in section 2.2 [20-23]. All the calculated fatigue lives of the bridge are shown in Table 6.

Table 6. Fatigue lives of the bridge girder

Fatigue load model	Remaining fatigue life (years)			
	Miner's rule		Non-linear damage model [section 2.2]	
	Uncorroded member	Corroded member	Uncorroded member	Corroded member
FLM4	284	94	143	85
AFLM-Scenario 1	681	146	343	108
AFLM-Scenario 2	318	109	160	90
AFLM-Scenario 3	181	93	91	82

5.3 Results and discussion

Stress analysis results shows that that bridge exposed to alternative stresses and vulnerable to corrosion fatigue as there are reported corrosion damage and it is in corrosive marine environment. Table 6 shows that there is significant change of fatigue lives depend on the method that used for analysis and life assessment. A 49.65% and 9.57% difference can be seen when comparing Method 1 and 3 for both uncorroded and corroded members respectively. There are 49.63%, 49.69% and 49.72% differences when compare Method 2 and 4 for Scenario 1, 2 and 3 respectively for uncorroded members. Similarly, 26.03%, 17.43% and 11.83% differences can be observed when compare Method 2 and 4 for Scenario 1, 2 and 3 respectively for corroded members. Comparison of all the methods has numerous uncertainties associated with fatigue life evaluation methods that may lead to different outcomes.

6 Conclusions

Remaining fatigue life evaluation of a specific road steel bridge was performed by using, (i) two different fatigue load models (i.e. FLM4 and AFLM), (ii) simulating structural degradation based on the reported corrosion damage (i.e. stress analysis using reduced cross section and using recently proposed S-N curve of corroded de-tails), and (iii) using conventional Miner's damage rule and recently proposed non-linear damage accumulation model. The fatigue lives calculated based on above methods were compared and discussed.

Results shows that Method 4-scenario 3 gives the lowest remaining fatigue life which is 82 years under the condition that there will not be any increase of loading, and/or any change of current corrosion state and corrosive environment. This indicates the using scenario 3 (i.e. AFLM with 15% heavy vehicles) of the discussed alternative fatigue load model with non-linear fatigue damage accumulation model provides conservative prediction to remaining fatigue lives for both corroded and uncorrected states. The differences of calculated fatigue lives between each method, emphasize the need for precise S-N curves which represent degraded strength state of the critical detail, realistic load models and an accurate fatigue damage accumulation model for aging bridges to implement conservative judgement remaining life. In addition, the actual traffic load measurements can be considered to provide more accurate response ranges.

Consideration could be given to a comprehensive examination of the bridge's cur-rent condition to propose strategies for life-extension and bridge maintenance. In addition, measurements of the actual traffic load can be undertaken to provide more accurate bridge response ranges.

References

[1] Adasooriya N. D, Pavlou D and Hemmingsen T 2020 Environment-assisted corrosion dam-age of steel bridges: a conceptual framework for structural integrity, *Corrosion Reviews*, 38, no. 1, 49-65.

[2] Nada Al-Eidan. What are the most dangerous types of corrosion and does the dangerous vary depending on the type of metal?, 2015.

[3] Gibson stainless & specialty INC. Corrosion types and prevention, 2017.

[4] Revie R.W: 2008 *Corrosion and corrosion control an introduction to corrosion science and engineering.* John Wiley & Sons.

[5] Mohammad Al-Emrani and Mustafa A 2014 *Fatigue design of steel and composite bridges.Göteborg*: Chalmers Reproservice.

[6] Griggs-Jr F: Eads Bridge at St. Louis, 2017 *Structure Magazine*, 18-22.

[7] Kreislova K, and Geiplova H 2012 Evaluation of corrosion protection of steel bridges, *Proce-dia Engineering* 40, 229-234.

[8] Kulicki J M, Prucz Z, Sorgenfrei D. F, and Mertz D R 1990, Guidelines for evaluating cor-rosion effects in existing steel bridges, *Transportation Research Board*, NRC, Washing-ton, D.C.

[9] Adasooriya N D, Pavlou D, and Hemmingsen T 2020, Fatigue strength degradation of cor-roded structural details: A formula for S-N curve, *Fatigue Fract Eng Mater Struct*., 43(6), 1199-1213.

[10] Adasooriya N. D, Pavlou D, and Hemmingsen T 2020, S-N curve for riveted details in corro-sive environment and its application to a bridge, *Fatigue Fract Eng Mater Struct*., 43(4), 721-733.

[11] Adasooriya N D 2020, Structural integrity of steel bridges: Environment-assisted cracking.

[12] Aeran A 2019, Life extension of offshore structures: A conceptual framework and fatigue damage models.

[13] NS-EN 1991-2:2003+NA:2010. Eurocode 1: Actions on structures part 2: Traffic loads on bridges.

[14] Sandviknes J S S 2021, Environment-assisted fatigue of steel bridges: A conceptual frame-work for structural integrity/life assessment. *Master's thesis*, uis, (2021).

[15] Sæstad A, Ersdal G and Siriwardane S. C 2016, Comparison of deterministic and probabilistic fatigue assessment methods: A case study of a road bridge, *Proceedings of the IABSE Congress Challenges in Design and Construction of an Innovative and Sustainable Built Environment*, September 21-23, Stockholm, Sweden,275-283.

[16] Aeran A, Siriwardane S.C, Mikkelsen O, and Langen I 2017, An accurate fatigue damage model for welded joints subjected to variable amplitude loading. *In IOP Conference Se-ries: Materials Science and Engineering*, 276, 012038. IOP Publishing.

[17] NS-EN 1993-1-9:2005 + NA: 2010.Eurocode 3: Design of steel structures – part 1-9 Fatigue

[18] Siviero E. and Pavan R 018, Assessment of existing steel bridges: codes and standard, *IOP Conference Series. Materials Science and Engineering*, 419(1).

[19] Siwowski T 2015, "Fatigue assessment of existing riveted truss bridges - case study," 63 (1), 125-133.

[20] Siviero E and Pavan R 2018, Assessment of existing steel bridges: codes and standard. *In IOP Conference Series: Materials Science and Engineering*, volume 419, 012006. IOP Pub-lishing.

[21] Oehme P 1989, Damage analysis of steel structures. *In IABSE Proceedings*, 139–189.

[22] Ye X.W, Su Y.H, and Han J 2014, A state-of-the-art review on fatigue life assessment of steel bridges. *Mathematical Problems in Engineering.*

[23] Siwowski T 2015, "Fatigue assessment of existing riveted truss bridges - case study," 63(1), 125-133.