



Faculty of Science and Technology

MASTER'S THESIS

Study program/ Specialization: Structural Engineering and Materials Science/Building Structures	Spring semester, 2014 Open / Restricted access
Writer: Askale Wendimu (Writer's signature)
Faculty supervisor: S.A.Sudath C Siriwardane External supervisor(s):	
Thesis title: Fatigue life estimation of existing steel structures under time-dependent structural degradation	
Credits (ECTS): 30	
Key words: <ul style="list-style-type: none">- fatigue life- pitting corrosion- remaining fatigue life	Pages: xi +92 + enclosure: 11 Stavanger, 29.06.2014 Date/year

Abstract

The purpose of this thesis is to present a method to estimate remaining fatigue life of steel bridge members subjected to time-dependent corrosion. This method contributes to the further development of the theoretical verification for the available methods and standards in remaining fatigue life of steel structures. The relevant standards for fatigue life and corrosion are evaluated, with focus on fatigue analysis, effects of the loss of the thickness of material due to corrosion and fatigue strength degradation of material due to corrosive environment.

A verification of steel bridge structural element in terms of fatigue life is very important for all old bridge construction. The fatigue strength of all structural steel depends on the environmental condition, the type of load and period. The bridge elements can reduce the thickness uniformly or partially. The main cause can be corrosion, where the structure can be subjected to different failure mechanisms during its lifetime. The main focus in this thesis is the introduce a method to estimate fatigue life of bridge members under time-dependent corrosion. Steel plates which are used as bridge elements are subjected to pitting corrosion. Because of this, the plates reduce the thickness irregularly. The plates be modeled and analyzed by using finite element analysis method employed computer program SAP2000. Verification of the introduced method has also done as an objective of this thesis.

In principle, the safety of construction generally depends on the design and considers the structural failure mechanism. The main causes of steel structure damage can be fatigue with corrosion and wear in most structural failure. A fatigue failure is one of the time dependent structural failures. Corrosion has a profound effect on the fatigue life of steel structure used in bridge construction.

This thesis includes verification of steel bridge members with respect to pitting corrosion and fatigue. To determine the effect of corrosion on a steel element, one must identify reduction of thickness in the material; this is read from the report of the physical research [\[29\]](#). Eurocodes and UK railway assessment code are mainly used as the basis for all calculations.

Keywords fatigue life; pitting corrosion; remaining fatigue life

Preface

This report constitutes the main project for master thesis "verification of a steel structure with respect to remaining fatigue life estimation of corroded steel bridge element" at the University of Stavanger in spring 2014.

The thesis is written by Askale Wendimu. I made contact with Associate Professor S.A Sudath C. Siriwardane at the University of Stavanger in hopes of getting a task. He was very positive and welcoming. After a few meetings I received a task to model and work on it. This means that I had to acquire more knowledge in computational corrosion in steel bridge structure and remaining fatigue life. In addition, during the project, I had to learn the modeling and calculation program Sap2000 that is essential to the task.

I would like to use this opportunity to thank my advisor Associate Professor S.A Sudath C. Siriwardane at University of Stavanger and the Professor Jasna B Jakobsen at University of Stavanger.

In addition, I would like to thank my family for supporting me in all condition in this work.

Symbols

C_B	is bulk concentration oxygen in sea/air
i_{gr}	is diffusion coefficients that depend on temperature
z	is the number of exchanged electrons
F	is Faraday constant
δ	is diffusion boundary layer
i_{corr}	is corrosion current density
I_{corr}	is corrosion current
D_{O_2}	is the diffusion coefficient of O_2
C	is a temperature independent preexponention
Q	is a temperature independent activation energy
R	is ideal gas constant
T	is absolute temperature
M	is the mole mass of the substance
ρ	is the density of the substance
ΔS	is the thickness reduction of the element per year
DOP	is a degree of pit corrosion intensity
n	is the number of pit
A_{pt}	is the surface area of the i^{th} pit
a	is the plate length
b	is the plate breadth
ΔK	is applied elastic stress intensity factor range
$\Delta\sigma$	is stress range
N	is number of cycle
S	is nominal stress
n_i	is actual number of cycles at stress range $\Delta\sigma$
N_i	is number of cycles to failure at constant stress range $\Delta\sigma$
γ_{Ff}	is partial safety factor for fatigue load
γ_{Mf}	is partial safety factor for fatigue strength

θ	is the angle of the crack plane
$\frac{1}{\sqrt{r}}$	r is the distance from the crack tip
σ_{eM}	is Von Mises yield stress
f_d	is the design yield strength
f_y	yield strength
γ_{M1}	material factor
K_f	is the fatigue notched factor
F_d	is the design stress
σ_{\max}	maximum applied stresses
K_t	is elastic stress concentration factor
σ_{nom}	nominal stress
N_x	axial force in x-axis
D_d	damage sum
max $(\Delta\sigma_{Ed,i})$	maximum value of the stress range for the design stress range spectrum $\Delta\sigma_{Ed,i} = \gamma_{Ff} \Delta\sigma_i$
$\Delta\sigma_D$	fatigue strength taken as the constant amplitude fatigue limit of the considered construction details
γ_{Mf}	partial factor for fatigue strength
λ	damage equivalent factor
γ_{Ff}	action effects partial factor for equivalent constant amplitude stress ranges for fatigue strength
$\Delta\sigma_c$	detail category
$\Delta\sigma_{E,2}$	equivalent constant amplitude stress range related to $2 \cdot 10^6$ cycles
$\Delta\sigma(\gamma_{Ff} Q_k)$	stress range caused by the fatigue load specified

n_{Ei}	number of cycles associated with $\Delta\sigma_{Ed,i}$ for band i in the design histogram
N_{Ri}	endurance (in cycles) obtained for a stress range of $\Delta\sigma_{Ed,i}$ considering the partial factors γ_{Mf} and γ_{Ff}

Notation

Crack propagation rate: Amount of crack propagation during one stress cycle.

Crack propagation threshold: Limiting value of stress intensity factor range below which the stress cycles are considered to be non-damaging.

S-N curve: Graphical presentation of the dependence of fatigue life (N) on fatigue strength (S).

Stress cycle: A part of a stress history containing a stress maximum and a stress minimum.

Stress intensity factor: Factor used in fracture mechanics to characterise the stress at the vicinity of a crack tip.

Stress range: The difference between stress maximum and stress minimum in a stress cycle.

Stress ratio: Ratio of minimum to maximum value of the stress in a cycle.

Table of Contents

Abstract	i
Preface	ii
Symbols	iii
Notation	v
Table of Contents	vi
List of figures.....	ix
List of tables	xi
CHAPTER 1 INTRODUCTION	1
1.1 Delimitation	2
1.2 Problem description	3
CHAPTER 2 CORROSION EFFECT ON FRACTURE OF STRUCTURES.....	4
2.1. Structural fracture	4
2.1. 1 Stress corrosion cracking.....	5
2.1.2 Hydrogen induced cracking.....	5
2.1.3 Wear failures	7
2.1.4 Corrosion	8
2.2 Types of corrosion	9
2.2.1. Galvanic corrosion	9
2.2.2. Selective corrosion	9
2.2.3. Crevice corrosion.....	9
2.2.4. Pitting corrosion	9
2.2.5. Uniform corrosion	9
2.2.6. Corrosion fatigue	10
2.3. Basic corrosion theory	11
2.3.1. Introduction	11
2.2.2. Theory of corrosion	11
2.2.3. General corrosion and pitting	13
2.4. Occluded chemistry of cracks and pit.....	17
2.5 Fatigue	18
2.5.1. Introduction	18
2.5.2. Fatigue load	19
2.5.3 Fatigue load for steel bridge	19

2.5.4. Fatigue damage.....	24
2.5.5. Environmental impacts.....	25
2.5.6. Fatigue design.....	26
2.6. Stress evaluation	30
2.6.1. Yield criteria.....	30
2.6.2. Von Mises yield criterion	31
2.6.3. Stress concentrations	31
2.6.4. Stress range.....	32
2.7. The theory of fatigue	33
2.8. The assumption of fatigue analysis	34
2.8.1. Fatigue limit.....	38
2.8.2. Verification of fatigue.....	38
2.9. Corrosion fatigue	41
2.9.1. Background	43
CHAPTER 3 PROPOSED FATIGUE MODEL FOR PITTING CORROSION	47
3.1. Effects of corrosion on fatigue of steel.....	47
3.2. Effects of corrosion on the fatigue failure equation	47
3.3 Stress concentration and fatigue strength	48
CHAPTER 4 VARIFICATION OF PROPOSED MODEL	51
4.1. General.....	51
4.2. Steel bridge structural elements verification	52
4.3. Basis of simulation	53
4.4. Control of steel bridges structural critical plates	54
4.5.1. Stress concentration factor	54
4.5. Dimension of the chosen plate.....	55
4.6. Actual stress	58
4.7. Stress concentration factor.....	62
CHAPTER 5 CASE STUDY	66
5.1. Steel member fatigue analysis method	66
5.2. Steel element fatigue strength analysis method.....	66

5.3. Time-dependent failure for steel bridge structural element.....	69
5.4 Factors that affect fatigue-life.....	69
5.5 Remaining fatigue life	70
5.6 Remaining fatigue life of corroded bridge member	71
5.6.1. Corroded bridge member.....	71
5.6.2. Stress evaluation.....	72
5.6.3. Fatigue life estimation	72
5.7. Remaining fatigue life	89
5.8. Explanation of remaining fatigue analysis in excel worksheet	89
CHAPTER 6 DISCUSSION, CONCLUSION AND FARTHER STUDIES	90
6.1. Discussion.....	90
6.2. Conclusion.....	91
6.3 Farther studies.....	91
References	92
APPENDIX	1
A.1. SAP 2000.....	1
A.1.1 General.....	1
A.1.2. Dimension of the chosen plate.....	1
A.1.3. Example of SAP 2000 model and analyses	2
A.2. Plate theory.....	5
A.2.1 Dimensioning of plates.....	8
A.2.2. Von Karman plate theory	10

List of figures

Figure 1 Charpy impact test machine [2]	5
Figure 2 Effect of Cu content on the amount of absorbed hydrogen under wet hydrogen sulfide environment [2]	6
Figure 3 Wear [20]	7
Figure 4 E/pH diagram for iron in water [2]	8
Figure 5 Example of pitting corrosion in a pipe [21]	9
Figure 6 Uniform corrosion [21]	10
Figure 7 General characteristics of corrosion fatigue curve [2]	10
Figure 8 Pitting corrosion deep growth in stainless steel [2]	14
Figure 9 Measuring depth of pitting deep (pit gauge) [22]	15
Figure 10 A corrosion pit and a crack exposed to an electrolyte [3]	17
Figure 11 Fatigue load model 3 [12]	20
Figure 12 Definition of the coordinate axis ahead of crack tip [3]	25
Figure 13 Dimensioning process for fatigue [1]	29
Figure 14 Example of stress concentration around a large hole [10]	31
Figure 15 Illustration of a definition of stresses and influence of tensile residual stresses [1]	32
Figure 16 Notched S-N curve [28]	36
Figure 17 Illustration over linear relationship	36
Figure 18 Illustration of the division of stresses and the number of cycles [4]	37
Figure 19 Stress range [1]	37
Figure 20 Cycle-dependent corrosion fatigue behavior [3]	44
Figure 21 Time-dependent corrosion fatigue behavior [3]	45
Figure 22 Cycle-dependent and time-dependent corrosion fatigue behavior in somebody's company [3]	45
Figure 23 Dimensions and pitting in specimen D2 for SAP2000 model [58]	55
Figure 24 Dimension and pitting in specimen F1 for SAP2000 model [59]	56
Figure 25 Dimensions and pitting in specimen F3 for SAP2000 model [60]	56
Figure 26 Dimensions and pitting in specimen G1 F3 for SAP2000 model [61]	57
Figure 27 Actual stress for plate 1	58
Figure 28 Actual stress for plate 2	59
Figure 29 Actual stress for plate 3	60
Figure 30 Actual stress for plate 4	61
Figure 31 Pit depths, radius, specimen thickness and stress concentrations for instrumented corroded plates [29]	62
Figure 32 Stress concentrations derived from strain gage data plotted vs. the pit depths in the plate specimens [29]	63
Figure 33 Stress concentration factor K_f in corroded plate specimen was derived from strain gage data and plotted vs. the function of plate thickness derived by pit radius. The correlation coefficient, r , for this data plot is +0.776 [29]	64
Figure 34 Stress concentration derived from FEM plotted vs. the pit depths	65
Figure 35 Fatigue strength curves for direct stress ranges [12]	67
Figure 36 Fatigue strength curve for shear stress ranges [12]	68
Figure 37 damage accumulation diagram [1]	70
Figure 38 Main truss girder [30]	71
Figure 39 General view of the riveted railway bridge [30]	71
Figure 40 Some of corroded location of the bridge [30]	71
Figure 41 50 S-N curve in air [10]	72

Figure 42 Stress range spectrum for the period 1885 – 1910.....	73
Figure 43 S-N curve for the year 1885-1910 by using Mathcad.....	74
Figure 44 Damage accumulation calculation for each year in the period 1911 -1930.....	75
Figure 45 S-N curve between years 1911-1930 by using Mathcad	75
Figure 46 Damage accumulation calculation for each year in the period 1931– 1950.....	76
Figure 47 S-N curve between years 1931-1950 by using Mathcad	77
Figure 48 Damage accumulation calculation for each year in the period 1951– 1970.....	78
Figure 49 S-N curve between years 1951-1970 by using Mathcad	78
Figure 50 Damage accumulation calculation for each year in the period 1971– 1975.....	79
Figure 51 S-N curve between years 1971-1975 by using Mathcad	80
Figure 52 Damage accumulation calculation for each year in the period 1976– 1985.....	81
Figure 53 S-N curve between years 1976-1985 by using Mathcad	82
Figure 54 Damage accumulation calculation for each year in the period 1986– 1995.....	83
Figure 55 S-N curve between years 1986-1995 by using Mathcad	84
Figure 56 Damage accumulation calculation for each year in the period 1996– 2001.....	85
Figure 57 S-N curve between years 1996-2001 by using Mathcad	86
Figure 58 Damage accumulation calculation for each year in the period 2001-now.....	87
Figure 59 S-N curve between years 1976-1985 by using Mathcad	88
Figure 60 Illustration of plate replaces [6]	10

List of tables

Table 1 Indicative number of heavy vehicles expected per year and per slow lane [12].....	21
Table 2 Set of frequent lorries [12]	22
Table 3 Definition of wheels and axles [12]	23
Table 4 Cumulative damage method [12]	40
Table 5 Pit depth, Von Mises & nominal stresses and stress concentration factor for corroded plates.....	63
Table 6 Damage accumulation calculation for the period 1885 – 1910.....	73
Table 7 Damage accumulation calculation for the period 1911 – 1930.....	74
Table 8 Stress range spectrum for the period 1931-1950.....	76
Table 9 Stress range spectrum for the period 1951 – 1970.....	77
Table 10 Stress range spectrum for the period 1971 – 1975.....	79
Table 11 Stress range spectrum for the period 1976 – 1985.....	81
Table 12 Stress range spectrum for the period 1986 – 1995.....	83
Table 13 Stress range spectrum for the period 1996 – 2001.....	85
Table 14 Stress range spectrum for the period 2001- now.....	87

CHAPTER 1 INTRODUCTION

The remaining fatigue life steel bridge is basically a bridge that "may be located near or over large fresh water lakes or major rivers, or be in close proximity to industrial pollution or even dust-laden atmospheres" [29]. Its main function is for transports that are light passenger cars, trucks and personal cars. The transports can have a weight from 8.9KN up to 355.8KN, and the bridge carrying these often.

Structural failure depends on the operating conditions, the use of material and the type of environment. For example fracture (brittle or ductile), plastic collapse, elastic instability, stress corrosion cracking, hydrogen induced cracking, corrosion, wear, fatigue and corrosion fatigue. The time dependent structural failures are stress corrosion cracking, hydrogen induced cracking, wear, corrosion, fatigue and corrosion fatigue. This thesis focuses on some of the time dependent structural failures, namely fatigue, corrosion and corrosion fatigue.

Old steel bridges designed by considering the traffic conditions at that time, have given service up to now, but the traffic has increased much more compared to the old days. Nevertheless the heavy vehicles become heavier than before. Therefore verification of the old steel bridge structure is necessary to determine and give the correct or almost exact remaining life of steel bridge. After the verification, one can give appropriate provisions for a more refined maintenance method, possible restriction of traffic and steel bridge rehabilitation.

The purpose of this task is to verify the corroded steel bridge members with respects to remaining fatigue life estimation. In this task should be performed the loss of plate thickness of the bridge elements and main truss subjected to time-dependent corrosion. Standards, like Eurocode 3 part 1-5 and part 1-9 and DNV etc, are mainly used as the basis for calculations. In the report, the plate checked with respect to fatigue and corrosion fatigue. To carry out the calculation necessary information from the document [29] from Illinois department of transportation is used. The plates are modeled and analyzed in the program Sap2000.

1.1 Delimitation

The delimitations of this thesis are:

- The steel bridge structural element verify only when it is in operation
- There is only performed remaining fatigue life
- There is not taken into in account all detailed design
- It is also included redaction of the plate thickness due to corrosion

In this thesis, one can see how engineering students solve structural problems, particularly how verify remaining fatigue life and how engineers give a solution for the corrosion fatigue structural failure.

Because time limitations may limit the task, so that it is feasible within the time limit that was given. The definition is done by recommendation from the internal advisors.

It isn't taken into account the effect of all types of load. For the design of steel bridge structure critical load cases must be identified, because it helps to limit the weight of vehicles.

The steel bridge structural elements/plates are modeled in SAP2000, instead of USFOS which was planned to use. The program use finite element analysis method (FEM) for analysis the structure. The structure is the considered as an assemblage of these elements connected at a finite number of joints called nodes points and the calculated result from this method is more accurate as compared to other method..

1.2 Problem description

A structure is a fundamental field referring to the recognition observation nature and parameter of patterns and relationships of entities. Structural engineers are the responsible persons for the detailed analysis and design of structures. There are two type of structures. Architectural structures like houses and buildings, and civil infrastructures like bridges and offshore structures. Architectural structures mostly work in close cooperation with an architect who will ultimately be responsible for design and also the strength of construction. Sometimes the architect is only responsible for the design and the strength done by structural engineers. Civil infrastructures are those structures that take a part in a loading role.

The principles of analysis and design of structures is based on the laws, regulations and standards. This helps to reduce the risk of human life and give a good safety of nation. In Norway there are planning and building laws that determine where and who can construct. Building regulations give the detailed provisions and standards give the requirements of construction in detail. All construction, both offshore and onshore, must have a good resistance for all type of load and have good strength.

The steel bridge member in this report "may be located near or over large fresh water lakes or major rivers, or be in close proximity to industrial pollution or even dust-laden atmospheres [29]. This demonstrates that the member is subjected to both fatigue and corrosion damage. It therefore must be designed to withstand all structural failure conditions. The bridge element shall have a capacity to withstand the normal load during operation and the effect of environment. This thesis task is thus to verify the steel bridge element with respect to fatigue life the effect of corrosion.

CHAPTER 2 CORROSION EFFECT ON FRACTURE OF STRUCTURES

2.1. Structural fracture

Collapses caused by physical failure of a construction can start from the design. Therefore, the design must consider all types of loads, from the material load up to in operation (the construction in service). The selection of material is also very important. Structural failure can refer to loss of the load-carrying capacity of a structural elements or the whole construction. The failure can be initiated when the material is stressed to its limit of strength, which can cause fracture.

Structural failure depends on the operating conditions, the using material and the type of environment. For example, fracture (brittle or ductile), plastic collapse, elastic instability, stress corrosion cracking, hydrogen induced cracking, corrosion, wear, fatigue and corrosion fatigue. The time dependent structural failures are stress corrosion cracking, hydrogen induced cracking, wear, corrosion, fatigue and corrosion fatigue.

Stress or strength is important criteria for design of failure of a structural member such as plate of any bridge. For calculation of stress value, the plate divided into the smaller of finite dimensions called finite element like rectangular and triangular. The study presented here is the plate as structural element subjected to fatigue load, corrosion fatigue and pitting corrosion is considered.

2.1. 1 Stress corrosion cracking

Stress corrosion cracking is the cracking of a metal induced from the combined influence of tensile stress, temperature and corrosive environment. The problem itself can be quite complex and it is common throughout industry. The cracks grow in silence. Stress corrosion cracking attacks the passive materials where no uniform corrosion takes place. To measure the toughness of metals one could use the Charpy testing [2] that determines the impact energy of materials.

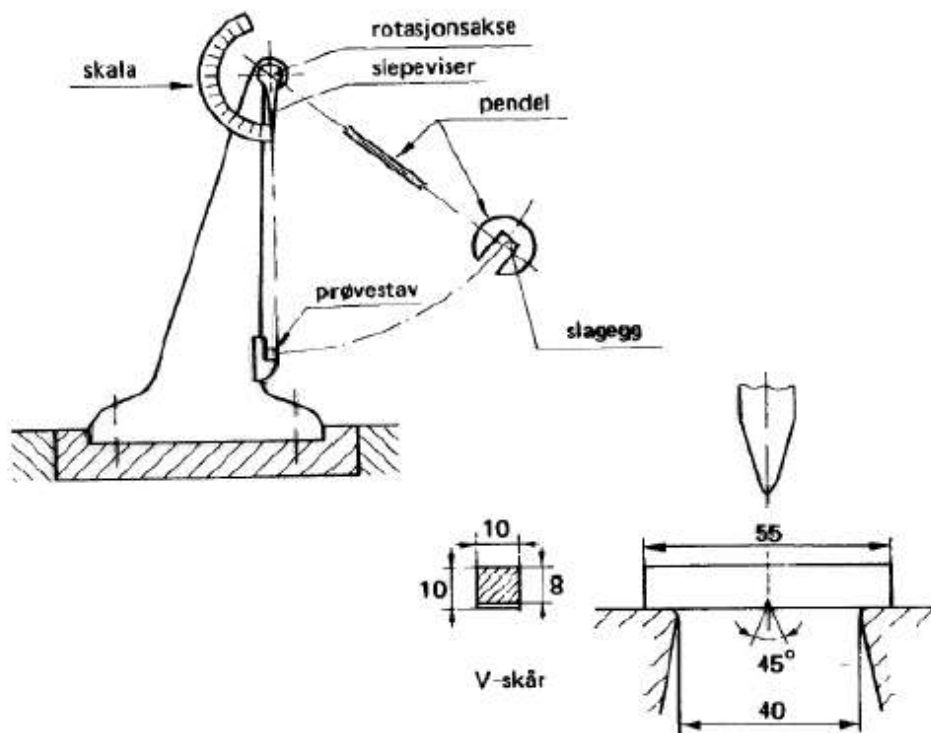


Figure 1 Charpy impact test machine [2]

2.1.2 Hydrogen induced cracking

Hydrogen induced cracking (HIC) is internal cracking in material by trapped nascent hydrogen atoms. $2H_{\text{atoms}} \rightarrow H_{2 \text{ gas}}$

If the formation of molecular hydrogen is suppressed, the nascent atomic hydrogen atom can diffuse into the interstices of the metal instead of bending harmlessly evolved as a gaseous reaction product thus producing a crack in the material. [2] Nascent hydrogen is purported to consist of a chemically reactive form of hydrogen that is freshly generated.

Hydrogen induced cracking doesn't need external force to get cracked. If H_2S is present the $H+H \rightarrow H_2$ will be poisoned. Hydrogen Induced Cracking (HIC) can be avoided by reducing the amount of H entering the steel. This can be:

1. Reduce corrosion. Dehydrating, always be above the dew point
2. Using corrosion inhibitors, film forming amines
3. Reduce S content in the steel
4. Add 0.25% - 0.3% Cu to the steel, see fig. 2.
5. Presence of an oil film has a good effect

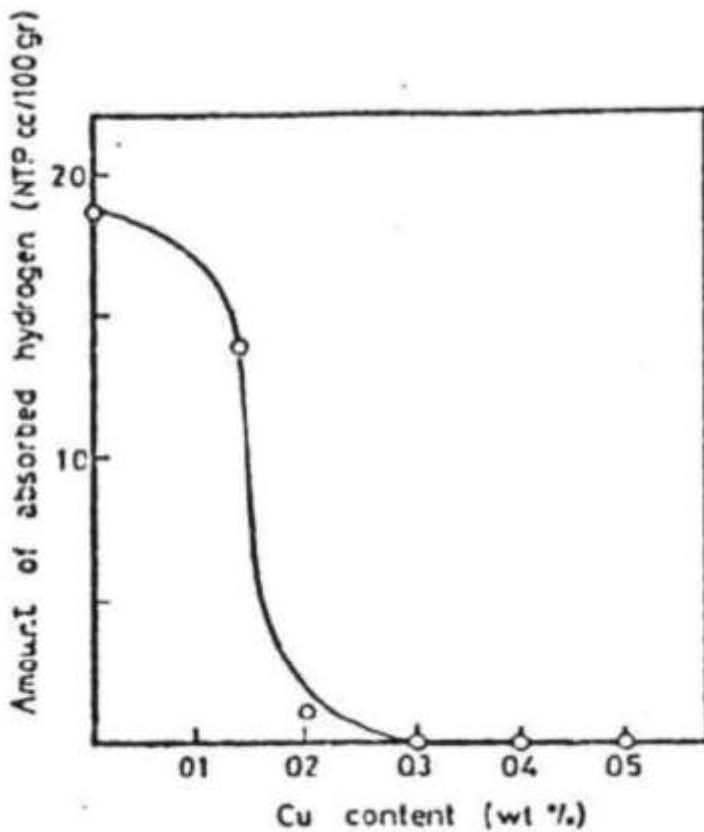


Figure 2 Effect of Cu content on the amount of absorbed hydrogen under wet hydrogen sulfide environment [2]

2.1.3 Wear failures

Wear failure is solid surface damage where there is loss of material from a solid surface. The mechanical action of a contacting element causes a gradual degradation that can lead to displacement of material or destroy a material by friction. “Wear is related to interactions between surfaces and more specifically the removal and deformation of material on a surface as a result of mechanical action of the opposite surface”. [\[20\]](#) The material can retire in partial, just on a place that has a physical touching.

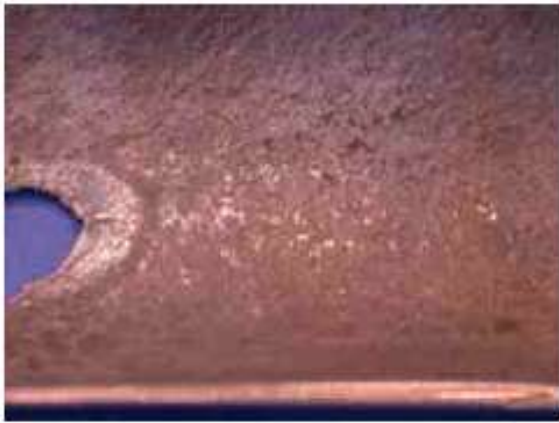


Figure 3 Wear [\[20\]](#)

2.1.4 Corrosion

Corrosion is a result of electrochemical reduction reaction. Theories of corrosion divide into two, thermodynamics and the kinetics of corrosion reactions. Thermodynamics clarify either a reaction is theoretically possible or not, and if a metal in aggressive environment can corrode or will not corrode. This can be done by using Pourbaix diagram. The kinetics of corrosion reactions explains how fast a reaction goes under certain conditions and the speed of corrosion.

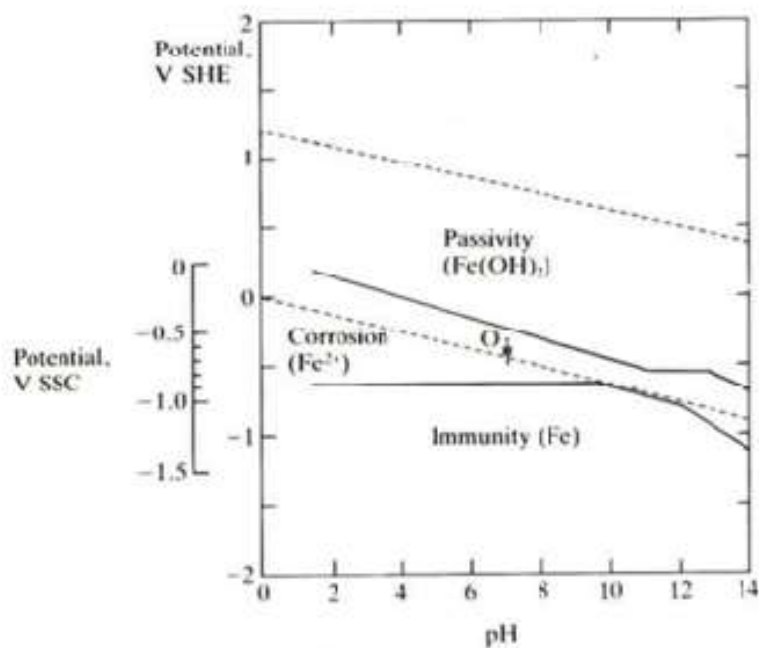


Figure 4 E/pH diagram for iron in water [2]

2.2 Types of corrosion

2.2.1. Galvanic corrosion

Galvanic corrosion is cathode efficiency and environmental corrosion. Galvanic corrosion cracks increased at contact surfaces between two metal articles.

2.2.2. Selective corrosion

Selective corrosion occurs at preferred sites on a metal surface, corrosive attack on structural constituents.

2.2.3. Crevice corrosion

Crevice corrosion rate can be similar to pitting. This attack occurs because part of a metal surface is in a shielded environment, compared to the rest of the metal that is exposed to a large value of electrolyte.

2.2.4. Pitting corrosion

Pitting corrosion is initiation on the surface at weak metallurgical points as defects, damages, etc. it causes localized scarring. The basis metal is eaten away and perforated in places in the manner of holes, the rest of the surface being affected only slightly.



Figure 5 Example of pitting corrosion in a pipe [21]

2.2.5. Uniform corrosion

Uniform corrosion is that the reaction starts at the surface and proceeds uniformly. It means a corrosion attack with constant dissolution rates over the whole surface. It is the most common and it takes place on low-alloyed steel. The damage mostly found in water tubes, buildings, bridges etc. This is caused by not using a proper material, not considering this kind of corrosion on the design, and not using protections /coating.



Figure 6 Uniform corrosion [21]

2.2.6. Corrosion fatigue

Corrosion fatigue is the result of simultaneous action of a cyclic stress and chemical attack. Corrosive environments have influence and a fatigue life of a construction becomes short. The type of material can be the cause of the failure. The same material would corrode in a normal environment. Small pits can be a result of chemical reactions between the environment and material. “There are many similarities between corrosion fatigue and stress corrosion crack, but the most significant difference is that corrosion fatigue is extremely non - specific. Corrosion fatigue crack growth rate is faster in seawater than in air.”

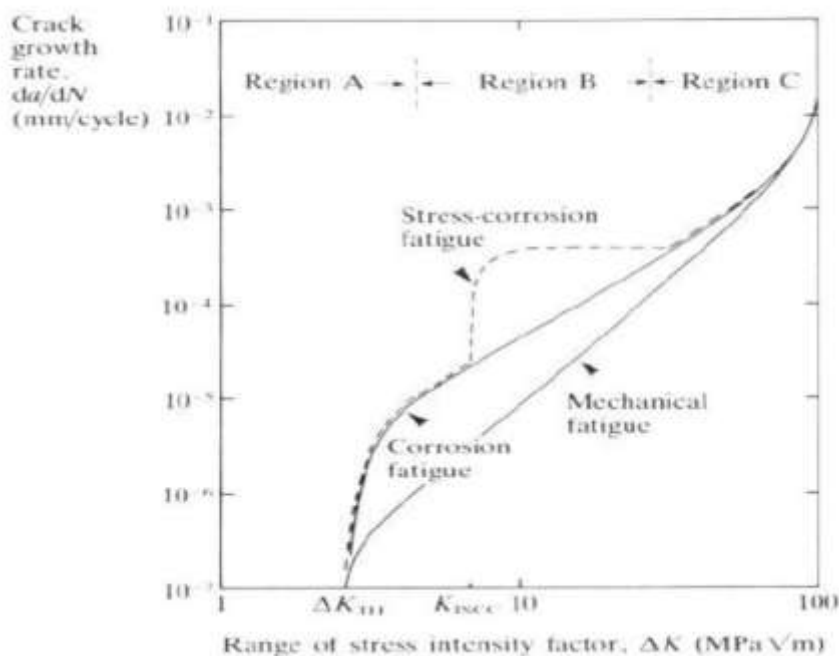


Figure 7 General characteristics of corrosion fatigue curve [2]

2.3. Basic corrosion theory

2.3.1. Introduction

The main degradation mechanisms of a surface are corrosion, erosion, wear, fatigue and chemical deterioration. In this document, these degradation mechanisms will be described focusing on those for surfaces and cracking. The corrosion theory discussed in this thesis is based on the subject of “Corrosion MOA 160”.

2.2.2. Theory of corrosion

Corrosion is a chemical reaction of a metal with its environment as a result of which electric current flows. All the environments which have been considered in the experiments have been aqueous, but this does not mean that corrosion is absent when water is absent. Many corrosion reactions occur in what can be considered as dry environments.

Generally, two important conditions must be present in order for a corrosion reaction to develop:

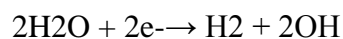
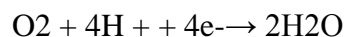
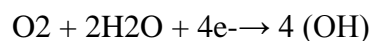
1. Water must be present
2. It must take an anodic and cathodic reaction at the same time

As an example we can see the corrosion of iron in acidic or neutral / alkaline electrolyte

Anodic reaction (oxidation): $\text{Fe} \rightarrow \text{Fe}^{2+} + 2\text{e}^-$ [2]

An anodic reaction is the electron-producing reaction, called an oxidation process. According to the simplified equation, the metal is being degraded into ions. To identify different regions of a corroding metal/electrolyte interface at which the electrodic processes occur. If the reaction are net anodic, that part can called the interface an anode.

Cathodic reaction (reduction): $2\text{H}^+ + 2\text{e}^- \rightarrow \text{H}_2$ (gas)



The cathodic reaction is possible when the electrons released can pass to a site on the metal surface. The rates of anodic and cathodic reaction must be equivalent according to Faraday's laws, been occur during a cathodic reaction:

- A given species undergoes reaction, in other words, there is a decrease in its oxidation number.

- A gain of electrons at the cathodic site, as shown in a cathodic reaction. [2]

That reaction however is just a half-cell reaction, which means it is usually coupled, or it happens, along with an anodic reaction. During electrolysis reaction moves towards cathode where it accepts the electrons and becomes reduced

There are basically three main ways to enter the corrosion rate. The most direct practical interest to thickness reduction per. unit time is most appropriate. Another way, albeit a related method, is to set the corrosion rate by finding the weight loss pr. surface and unit of time. That one can enter the corrosion rate by corrosion current density shown in:

$$i_{corr} = \frac{I_{corr}}{A} \quad (2.1)$$

The corrosion rate is the amount of metal ions removed from the metal date per unit area and unit time. This ion transport can be specified as electric power per. unit area, i.e. the corrosion current density. The concept of corrosion current density is directly useful and practical in connection with the cathodic and anodic protection.

Temperature has a lot of impact on the corrosion development. In practice one often has multiple cathodic reactions taking place in parallel, allowing the exchange current density and the limit current density to be different. Generally the temperature will affect both the exchange current density and Tafelgradient, but by oxygen corrosion temperature will have the greatest impact on the diffusion limit current density. On surfaces that are free of the diffusion barrier coating, the diffusion limit current density is given as:

$$i_{gr} = \frac{zFD_{O_2}}{\delta} C_B \quad (2.2)$$

The effect of temperature is given by the diffusion coefficient D dependence on temperature, which describes using Arrhenius equation [2]:

$$D_{O_2} = C \exp\left(-\frac{Q}{RT}\right) \quad (2.3)$$

It is possible to reduce the damage of corrosion on steel structure, by using corrosion protection. There are many corrosion protections that are very useful to minimize the corrosion damage. To name some:

Cathodic protection, which is a system of corrosion prevention that turns the entire pipeline into the cathode of a corrosion cell, is used extensively on steel pipelines in the oil and gas industries. The two types of cathodic protection systems are galvanic and impressed current.

Cathodic protection is lowered the electrochemical potential of the metal surface into the immune neighborhood Pourbaix diagram. In practice, there is so much external power applied that the Andean iron dissolution is very slow.

2.2.3. General corrosion and pitting

Normally there are two types of corrosion damage which can reduced the thickness of the plate, namely general/uniform and pit corrosion. Uniform corrosion is where; the thickness of the entire plate is reduced uniformly. In the case of pitting corrosion, the plate thickness reduced in localized regions.

General/uniform corrosion is a result in a fairly uniform penetration over the entire exposed metal surface. As the name implies it attacks the entire surface. It represents the greatest destruction of metal on a tonnage basis. The uniform corrosion is not of great concern from a technical standpoint, because the life of a structural element can be accurately estimated on the basis of comparatively simple immersion tests. These tests allow weight loss to be monitored, and the reduction of thickness as a function of time can be calculated.

One can use Faraday's laws easily calculate the general corrosion on a structure by dimensions thickness reduction per year:

$$\frac{ds}{dt} = \frac{i_{corr} \cdot M}{(z \cdot F \cdot \rho)} \quad (mm/s) \quad (2.4)$$

$$\Delta S = \frac{i_{corr} \cdot M \cdot 31.5 \cdot 10^6}{(z \cdot F \cdot \rho)} \quad (mm/\text{\AA}r) \quad (2.5)$$

For example iron, by inserted in equation [5.5] $M_{Fe} = 56g/mol$, $\rho = 7.8Kg/dm^{-3}$, $z = 2$ for $i_{corr} = 200mA/m^2$ get one that thickness reduction per year will be:

$$\Delta S = \frac{200mA/m^2 \cdot 56 \cdot 10^{-3} Kg/mol \cdot 31.5 \cdot 10^6 s/\text{\AA}r}{(2 \cdot 96500As \cdot 7.8 \cdot 10^{-3} Kg/m^{-3})} \cong 0.2mm/\text{\AA}r$$

This design value of current density for cathodic protection at start of temperate zone is for a depth less than 30mm.

General corrosion modest has a rate of approx. 0.2mm / year that is roughly calculated 2 mm over a 10-year period. One would therefore assume that by dimensioning the extra plate thickness corrosion problems can be controlled.

Nevertheless, the realities are that corrosion attacks are very unevenly distributed. The plastic zone, for example, speed can be much higher, in the range 0.5-1.5 mm / year. When the temperature on the structural elements increases, the corrosion rate increases. In presence of chloride ions pits are growing by autocatalytic mechanism. Pitting corrosion can be measured by using dial gauge. The actual pitting corrosion phenomenon is shown on propeller shaft of high speed craft. Moreover, the Cl-ions in seawater breaking down the oxide film which protects the material. This local degradation creates a small anode surrounded depends a large cathode.

Pitting corrosion is a highly localized form of corrosion that produces sharply defined holes. It is one of the most insidious forms of corrosion and can cause failure by perforation while producing only a small weight loss on the steel construction material.

According to ASTM standard pitting corrosion can test by Ferric Chlorde pitting test.

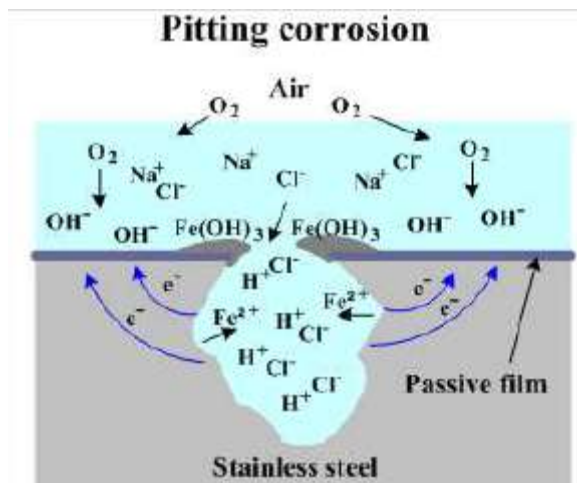


Figure 8 Pitting corrosion deep growth in stainless steel [2]



Figure 9 Measuring depth of pitting deep (pit gauge) [22]

All engineering construction steel is susceptible to pitting. Pitting occurs when one area of steel becomes anodic or if the steel element comes in contact with highly localized corroded metal. This is the cause accelerated localized attack.

It is possible to minimize pitting corrosion damage by:

- Reducing the aggressiveness of the environment by avoiding chloride ions concentration, oxidizing agent and temperature
- Upgrading the materials of construction by using highly quite construction material like stainless steel, duplex steel etc
- Modifying the design of the system by using appropriate geometrics

Pitting corrosion attacks initiate on an open surface. Pitting corrosion is influenced by many different parameters like:

- The environment
- Metal composition
- Potential
- Temperature
- Surface condition

There are phenomenological aspects of pitting corrosion that include the stochastic nature of the processes and the stages of localized attack like passive film breakdown, stable growth, etc. to name some local attack. Passive film breakdown that occurs in the presence of aggressive anionic species and chloride ions could be the cause of classical pitting corrosion.

The rate of pitting tends to vary with the logarithm of the bulk chloride concentration. Because chloride is a negatively charged ion of a strong acid and many metals are positively charged ions they exhibit considerable solubility in chloride solutions.

Pitting corrosion is characterized by the depth of penetration and localized form of corrosion. It is a very difficult type of corrosion, which has unknown mechanisms. Steel pipes are easily affected by pitting corrosion. Stainless steel is less susceptible to pitting corrosion, maybe if exposed to saline environment. There are three mechanisms of pitting stage, which are initiation of pitting, propagation of pitting and pitting termination.

Initiation of pitting occurs when there is one plate, on the plate surface there are oxygen and sodium chloride solution, and the metal is passive. So there are same conditions in addition like seawater and oxygen dissolving. In general, a corroded material found grain boundary and double grain boundary zone that has higher tendency for dissolution. Now there are same place in grain boundary zone or double grain boundary zone, which have higher energy that compare with the system. So when it dissolves oxygen in water, cathodic reaction $O + 2H_2O + 4e^- \rightarrow 4(OH^-)$ and anodic reaction takes a place. All throughout the surface those two reactions are happen. And then the solution is stagnant in nature so in pit forms. Since there are high energy zone there could be momentarily higher corrosion at particular region. At some moment there is a section where have extra dissolution. This is the start of the pitting corrosion. There is Fe^+ in all surfaces and oxygen's is come in and forms OH^- then the situation gradually changing, since it dissolve oxygen and

Propagation of pitting occurs from the start of pit the biggest area is cathode and small area is anode. This gives an unfavorable area ratio where more and more anodic dissolution takes place in the anodic zone. So that gradually in the section there is a higher metal ion concentration, which acts like electro positive sector. Then the system will try to neutralize the situation, since there is sodium chloride in the system, and the chloride ion will try to

come into the anodic zone. The content of hydrogen is being high in anodic zone and the value pH smaller, break in passive layer. The situation becomes autocatalytic nation.

The assessment of pitting corrosion can done by degree of pitting intensity (DOP). It defined as the ratio percentage of the corroded surface area to the original plate surface area:

$$DOP = \alpha = \frac{1}{ab} \sum_{i=1}^n A_{pi} \cdot 100(\%) \quad (2.6)$$

2.4. Occluded chemistry of cracks and pit

The result of occluded volumes in a different local chemistry at electrochemical reactions can influences subsequent reactions. Occluded environment at the bottom of a pit and the tip of a crack differs from the bulk environment is illustrated schematically. [3] The occluded chemistry at the bottom of the pit can lead to initiation of environmental cracks. The occluded chemistry at the tip of an environmental crack often has a significant effect on the propagation rate.

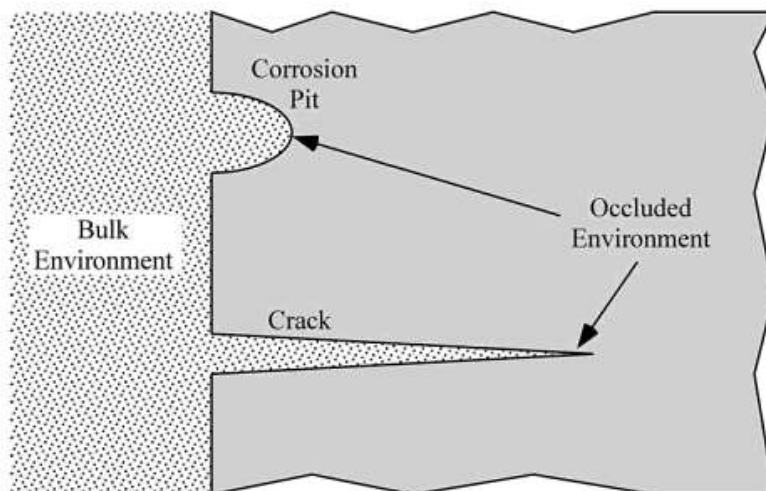


Figure 10 A corrosion pit and a crack exposed to an electrolyte [3]

2.5 Fatigue

2.5.1. Introduction

Fatigue is a failure mechanism, which attacks a structure when it subjected to dynamic and fluctuating stresses. Wohler performed the first systematic investigation of fatigue as a fracture form around the year 1860. The starting point was a series of seemingly incomprehensible fracture of axles in the railway at load that were far lower than the material static strength. A fatigue failure is possible to occur at a stress level considerably lower than the tensile or yield strength for a static load. Fatigue fracture characterized by a stable crack growth under variable load, which leads to fracture by a randomly high load on a reduced cross sectional area.

In general, the main causes of steel structure damage can be fatigue with corrosion and wear, in most structural failures. Fatigue failures occur due to action of fluctuating stress that the structural element subjected to repeated cyclic loading. Its mechanism generally shows in the form of developing cracks at particular location. The construction detail subjected to repeat cyclic loading can lead to progressive damage and it is represented by a loss of resistance with time. This kind of failure called fatigue failure.

The crack of fatigue hardly ever occurs in the base material but the corrosion occurs remotely from plate construction detail, this can be the same for any type of construction element.

Fatigue can be characterized as the notion of a material becoming tired, a material failing by lose of its nominal strength. This can be caused by cyclical stress, cyclic loading stress limit less than static stress capability.

One of the main causes of the structural material damage fatigue can occur due to action of fluctuating stress. On the part of material at the particular locations, fatigue come into sight in the form of a crack. Fatigue cracks usually start off at the surface. Once a crack is initiated, it grows slowly as the stress cycles are repeated. In the point of view of fatigue, the connection of any structural members can be critical place. Fatigue cracks in deck, bridge or any other construction, which subjected to cyclic load, could get cyclic actions and stresses as the result of the load.

Existing construction subjected to damage with respect to the long service time. This is normal; the challenges can be different from structures to structures. It depends on the use of material, the environmental condition and the geological condition. For analysis the existing structure, there are many needs, like all documents, which tell from the design up to installation. And also there was any modification or maintenance or use of material. It is necessary in terms of to continue verification of design and analysis. The design and analysis requirement of existing construction and the new one is the same. It verified by using the new standards and codes, the evolution of the strength of material and the load variation are checked in terms of today.

2.5.2. Fatigue load

A structure subjected to many types of loads during its lifetime. All those loads are different in magnitude and direction, which caused to stress variation and can lead to fatigue damage. Those loads are permanent, live load, environmental load and died load. In practice, the environmental load most commonly causes fatigue damage for both type of structures, in the building and offshore structures. The load is varying and can be big on the same area, which gives high stress on a local structural element. Local stresses for fatigue design need to be determined after accounting the temporal and spatial variations of load. For welded structures, the main parameter to represent the variation in time is the stress rang.

In general the purpose of designing against fatigue is to avoid failure problems for reasons of safety, economy, durability and liability.

2.5.3 Fatigue load for steel bridge

As considered in Eurocode [\[12\]](#), there are many load models, which are used for road bridges. For fatigue of road bridge design, "traffic running on bridges produces a stress spectrum which may cause fatigue. The stress spectrum depends on the geometry of the vehicles, the axle load, the vehicle spacing, the composition of the traffic and its dynamic effects [\[12\]](#) take in account in general. For bridge structures have five fatigue load models that denoted by FLM1 up to FLM5. The first three of them are considered to take the determination of the maximum and minimum stress resulting from the possible load arrangements on the bridge of

any of these models. The other two models proposed to determine stress range spectra resulting for the passage to lorries on the bridge.

Those models can be used to various form inceptions:

- to perform usual simple verifications
- to perform accurate verifications

According to Eurocodes, the models FLM 1 and FLM 2 are used to verify that the bridge lifetime is infinite regarding the fatigue phenomena. The models FLM 3, FLM 4 and FLM 5 are used to verify that the bridge has acceptable lifetime, which is logical for structural lifetime assumption with regarding the fatigue phenomena.

The fatigue load model 1 (FLM 1) derives from the principal characteristic load mode for bridge and has the values of the axle loads $0.7Q_{ik}$ and has the values of the uniformly distributed loads $0.3q_{ik}$. Due to the fact this load is the frequent SLS that is similar to those defined for the frequent load model.

The fatigue load model 2 (FLM 2) is set with a frequently idealized lorries that are defined by using tables. [12] Each lorry has a frequent axle load has a value Q_{ik} and be supposed to cross the bridge alone in the appropriate slow traffic lane. This helps to determine a maximum stress range, which ought to be compared to the CAFL.

The fatigue load model 3 (FLM 3) is simplified fatigue load model, which consists of a four axles single vehicle with a weight of $QE=120\text{KN}$ per axle. [12] This load model crosses the bridge in the middle line of the slow traffic lane. According to Eurocode statement, verification can be made with respect to finite fatigue life. There is a need for identify clearly a number of cycles that is indicated as a traffic category on the bridge. [12]

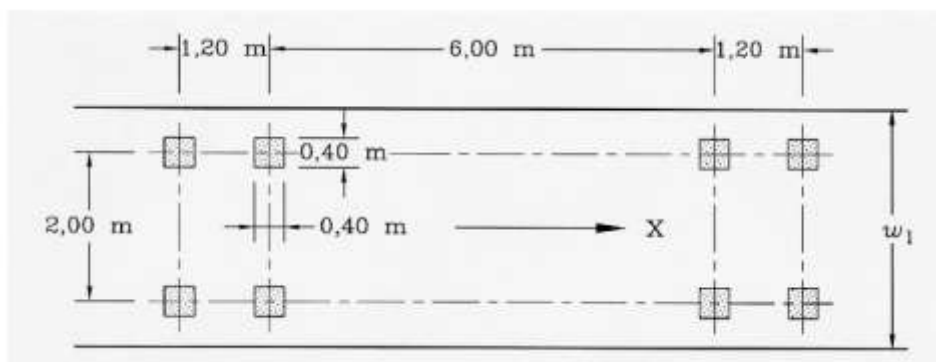


Figure 11 Fatigue load model 3 [12]

Traffic categories		N_{obs} per year and per slow lane
1	Road and motorways with 2 or more lanes per direction with high flow rates of lorries	2.0×10^6
2	Roads and motorway with medium flow rate of lorries	0.5×10^6
3	Main roads with low flow rates of lorries	0.125×10^6
4	Local roads with low flow rates of lorries	0.05×10^6

Table 1 Indicative number of heavy vehicles expected per year and per slow lane [\[12\]](#)






VEHICLE TYPE			TRAFFIC TYPE			
1	2	3	4	5	6	7
			Long distance	Medium distance	Local traffic	
LORRY	Axle spacing (m)	Equivalent axle loads (KN)	Lorry percentage	Lorry percentage	Lorry percentage	Wheel type
	4.5	70 130	20.0	40.0	80.0	A B
	4.20 1.30	70 120 120	5.0	10.0	5.0	A B B
	3.20 5.20 1.30 1.30	70 150 90 90 90	50.0	30.0	5.0	A B C C C
	3.40 6.00 1.80	70 140 90 90	15.0	15.0	5.0	A B B B
	4.80 3.60 4.40 1.30	70 130 90 80 80	10.0	5.0	5.0	A B C C C

Table 2 Set of frequent lorries [\[12\]](#)

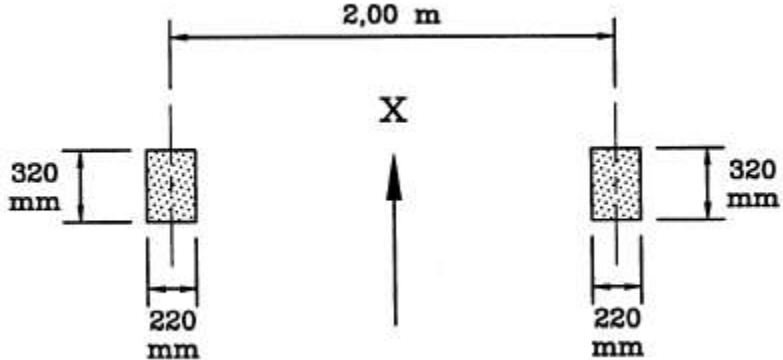
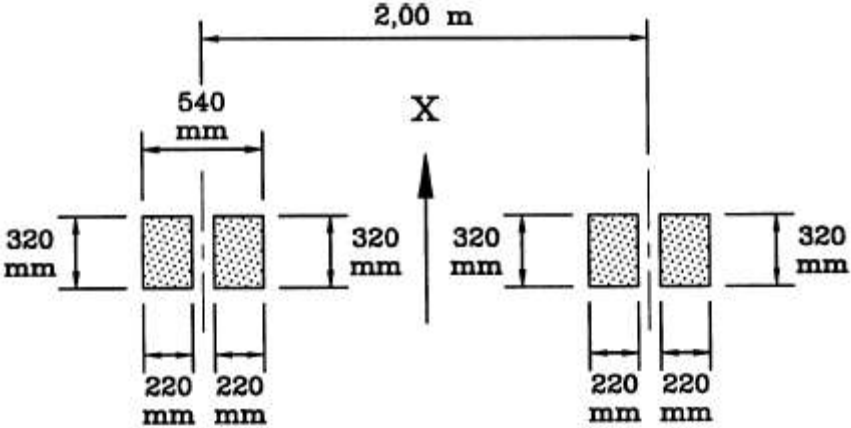
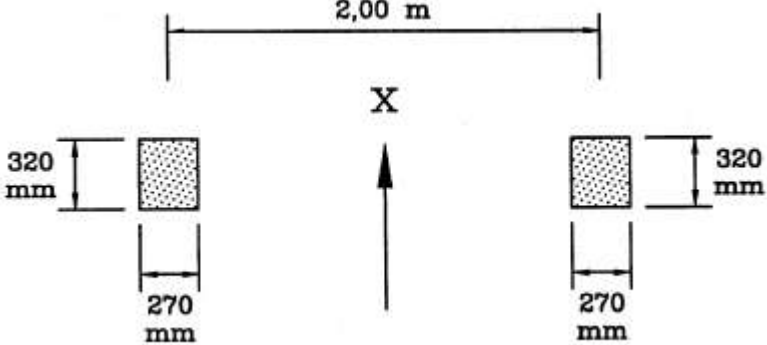
WHEEL AXLE TYPE	GEOMETRICAL DEFINITION
A	
B	
C	

Table 3 Definition of wheels and axles [12]

The fatigue load model 4 (FLM 4) has five equivalent lorries that each lorry is suspected to cross the bridge alone. This represent a certain percentage of the heavy traffic, it depend on the road type. These load models need the clear identification of N_{obs} . [12] The set of equivalent lorries for this load is given. [12]

Fatigue load model 5 (FLM 5) is associated with statistical tools, it start by identify and count the stress ranges then extrapolate the bridge fatigue life from short registered period. It is the general one.

2.5.4. Fatigue damage

Fatigue damage in a structural member could be due to several different loading conditions. The most common conditions will be that structural members are subjected to dynamic loads. There are basically two methods used to calculate the fatigue of structures. The simplest and most widely used method to date is based on finding the number of cycle, N , for different stress ranges, S . It is called S-N curves or Wöhler curves, where S within fracture mechanics terminology referred to $\Delta\sigma$. When one knows the S-N curve which is valid for the particular construction detail, then one knows the number of stress exchanges this detail can withstand at different stress level. The next step will be to determine how many exchanges at different stress level a structural detail will be exposed. One can then determine accumulated damage using Palm grain-Miner rule and one can estimate using this design lifetime.

Alternatively, one can use fracture mechanics to consider the life of a structure. Fracture mechanics tells the relation between the crack length a and the stress level $\Delta\sigma$, given the stress intensity ΔK front of the crack tip. That is this parameter which mainly determines whether the crack will start to grow and, then determines the speed of growth $\frac{da}{dN}$. An advantage of using fracture mechanics to estimate the structural details lifetime, one can determine the crack precursor critical points and phases to more easily determine unlucky mechanical and environmental effects and impacts.

According to T. L. Anderson [3], the prerequisite for applying fracture mechanics fatigue consideration is that the behavior of the crack tip could be uniformly defined by parameters, such as stress intensity ΔK . This means that for a material with different configurations a stationary crack will start to grow at the same critical tension intensity, provided there exists a singularity dominated zone crack tip.

This means that if one considering a system that is subjected to a constant stress intensity amplitude will form a plastic zone ahead of the crack tip, while the growing fracture will leave a relieved plastic backwater. If the size of the plastic zone ahead of the crack tip is substantially smaller than the elastic singularity dominant zone, the system will have a unified set of stress intensity.

The stress fields near a crack tip of an isotropic linear elastic material can be expressed as a product of $\frac{1}{\sqrt{r}}$ and a function of θ with a scaling factor K [3], crack tips produce a $\frac{1}{\sqrt{r}}$ singularity.

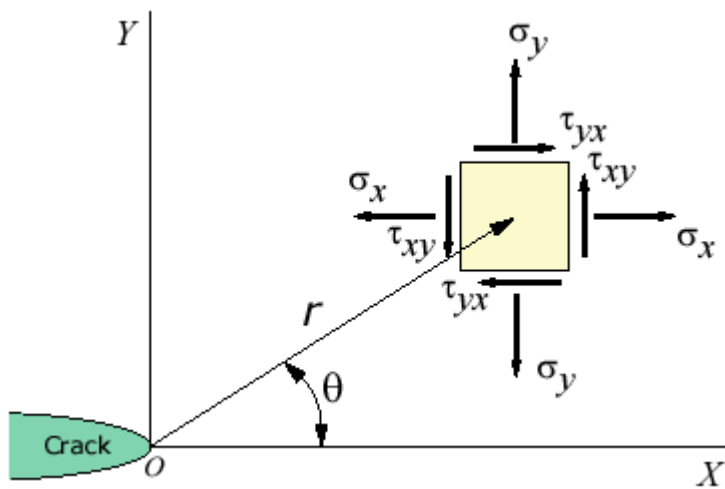


Figure 12 Definition of the coordinate axis ahead of crack tip [3]

In this thesis only the S-N curves method is used. As it is mentioned, it is simplest and most widely used method. It also cooperates with Physical research reports [3].

2.5.5. Environmental impacts

Environmental impacts of corrosion will generally reduce the life of the structural element. A corrosive environment will considering to the number of cycle to fracture decreases and limit is at least drastically reduced or eliminated, so that the fatigue fracture becomes possible by rather small stress. In the meantime, the number of cycles to fracture also depends on frequency and temperature.

Corrosion fatigue occurs for all material-environment combinations where one receives general or local surface corrosion. Therefore, it is standard to use cathodic or anodic

protection against the different structural members. Cathodic protection of structures is most used in the material that has more contact with water, and is used mainly to protect the steel surface. By applying coating, one prevents the steel from coming into direct contact with pure water or seawater and thus the steel does not disintegrate.

2.5.6. Fatigue design

The purpose of fatigue design is to ensure that the structure has an adequate fatigue life and can also form the basis for efficient inspection program during fabrication and the operational life of the structure. [1]

Steel bridge structural members (plates) that are in critical mode of fatigue failure shall be investigated with respect to fatigue. The design of steel plates can be carried out using two types of design methods according to Eurocode 3 [9]. Those are effective width method and reduced stress method. Comprising resistance models for shear force and transverse force in effective width method is very efficient for standard geometries. The reason for this is the method accounts not only for the post-critical reserve in a single plate element but also for load shedding between cross sectional elements. On the other hand, reduced stress method abstains from load shedding between cross sectional elements, but it fully accounts for the post-critical reserve in a single plate.

Local stresses for fatigue design need to be determined after accounting the temporal and spatial variations. For welded structures, main parameter to represent the variation in time is the stress range.

Von Mises Yield criterion expressed in x – y – z planes

$$\sigma_{eM} = \sqrt{\sigma_x^2 + \sigma_y^2 + \sigma_z^2 - \sigma_x\sigma_y - \sigma_y\sigma_z - \sigma_x\sigma_z + 3(\tau_{xy}^2 + \tau_{yz}^2 + \tau_{xz}^2)} \leq f_d = \frac{f_y}{\gamma_{M1}}$$

Basic design assumptions of fatigue

- Tensile residual stresses are always present
- All stress cycles effective drive the crack
- Spatial stress variation can be accounted for by using hot spot stress approach
- Global analysis of fatigue loading causes effects in member force
- Local analysis is carried out to determine hot spot stresses

- Fatigue load is caused by dynamic loads, for marine structures primary source is wave load

As shown in the illustration [20], load effect starts with identifying and modeling as load that provide fatigue. Then continue with a static or dynamic analysis of the construction. Local stress concentrations in the potential points of crack growth are calculated, after which the stress scorched the long term distribution is determined. This provides information about the number of stress cycles n_i with a stress range S_i .

Wohler has developed the empirical methods; two of them are the stress-life approach and the strain-life approach. To characterize the important parameter for cyclic loading history are:

$$\text{Stress range} \quad \Delta\sigma = \sigma_{\max} - \sigma_{\min}$$

$$\text{Stress amplitude} \quad \sigma_a = \frac{1}{2}(\sigma_{\max} - \sigma_{\min})$$

$$\text{Mean stress} \quad \sigma_m = \frac{1}{2}(\sigma_{\max} + \sigma_{\min})$$

$$\text{Load ratio} \quad R = \frac{\sigma_{\min}}{\sigma_{\max}}$$

Stress-life approach: a plot can prepared like $\log(\sigma_a)$ versus $\log(2N_f)$ a linear relationship is commonly observed. Then the relationship between stress amplitude and lifetime will be:

$$\frac{\Delta\sigma}{2} = \sigma_a = \sigma'_f (2N_f)^b \quad (2.7)$$

The above equation [7.1] is called Basquin relation, this is valid only for zero mean stress . It modified by Morrow to taken accounts for mean stress effects. This wrote:

$$\sigma_a = (\sigma'_f - \sigma_m)(2N_f)^b \quad (2.8)$$

Whereas:

σ'_f is the true fracture strength for most metal

b is the fatigue exponent, Basquin exponent $\approx 0.05-0.12$

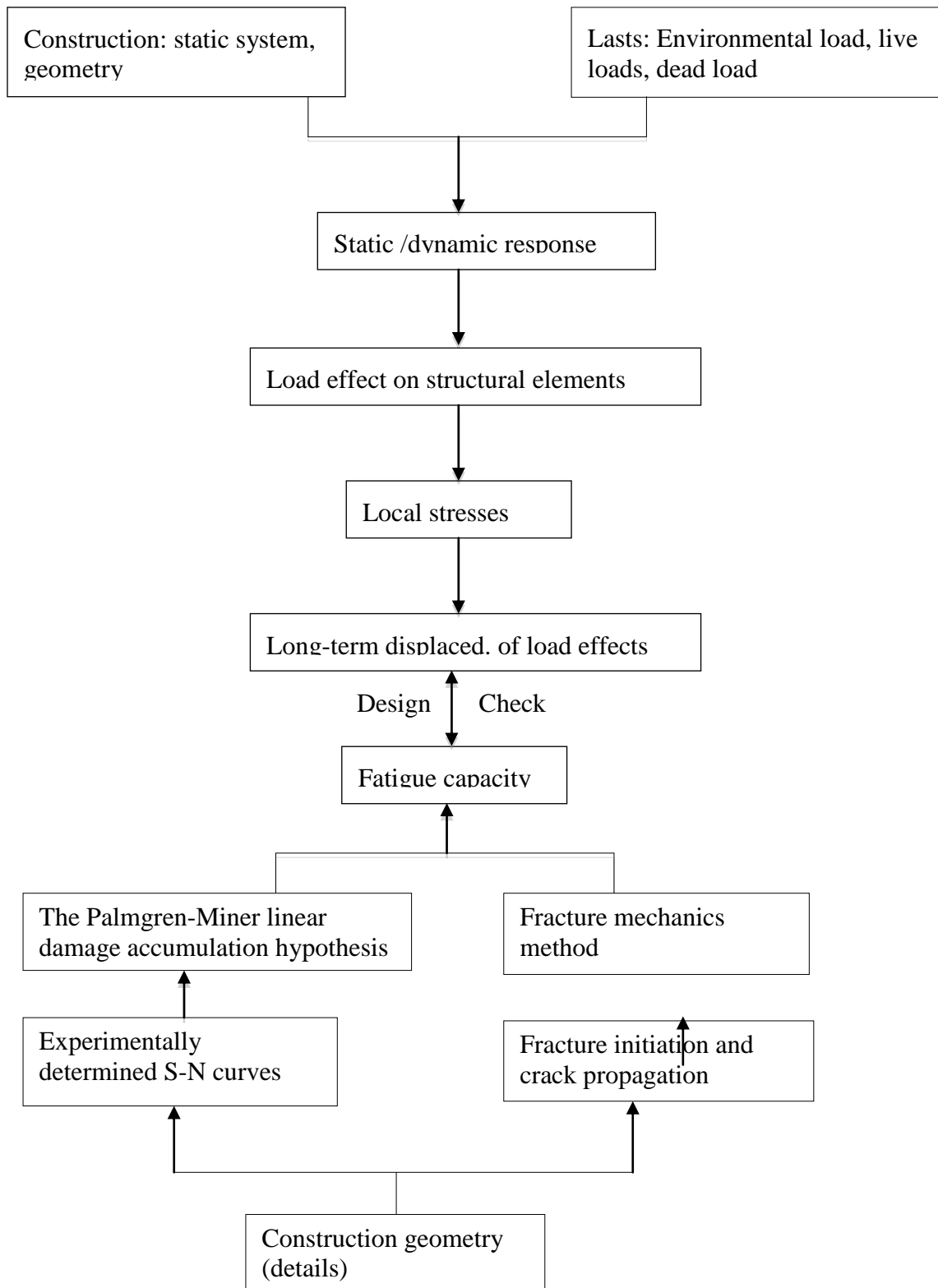
$2N_f$ is the number of reversal to failure

Stress-life approach is appropriate for situations involving primarily elastic deformation. For situations involving high stresses where there is significant plasticity, one can use the strain-

life approach. In this situation the loading is characterized by the plastic strain amplitude $\frac{\Delta\varepsilon}{2}$, then the following relationship come to picture:

$$\frac{\Delta\varepsilon_p}{2} = \varepsilon'_f (2N_f)^c \quad (2.9)$$

Figure 13 Dimensioning process for fatigue [1]



2.6. Stress evaluation

At an arbitrary section of a plate that is subjected to loads, there is an average force. In general, on in this section has shear forces, normal force, bending moment and torsion. Average forces that cause stresses in plates may be bending, axial and shear stress. The sum of all stresses in a plate shall not exceed the design stress F_d .

For classified and stress calculation of the critical structural elements, the load on a structural members should be calculated. Horizontal members are subjected to compressive forces where diagonal members are subjected to axial tinsel force. Commonly the diagonal members are within the critical structural zone, then by selecting or finding a nominal stress for the given member can get stress rang. For a weld structure, the details classification takes from the standard in terms of the type of welding and joint. Then stress is caused by the weld can take care by using S-N curves.

Most structural elements have some discontinuities like fillets, holes, etc, these discontinuities can be well defined geometrically. It is usually possible to determine a stress concentration factor, K_f , for those discontinuities in terms of the geometries. So that one can account for the local elevation of stress using the known relation between the local maximum stress and the applied nominal stress using:

$$\sigma_{\max} = K_t \cdot \sigma_{nom} \quad (2.10)$$

2.6.1. Yield criteria

The yield criterion ensures that it does not occur flotation in construction in ULS. The performers stress should be less than the yield stress divided by the safety margin. To avoid this, one must have sufficient dimensions and high enough yield strength of the material.

Mathematically formulated yield criterion as follows:

$$\sigma_{evm} \leq \frac{f_y}{\gamma_m}$$

There σ_{evm} is occurring effective stress (load effect) in construction, f_y is the yield strength and γ_m is the material factor.

2.6.2. Von Mises yield criterion

In a multi-dimensional stress state, it is not only the yield stress or fracture stress to be taken into account. It is also necessary to have criteria that describe the combination of normal and shear stresses providing yield. It should, in other words control when all voltages occur simultaneously. For practical design of steel structures is Von Mises criterion is commonly in use. [32]

2.6.3. Stress concentrations

The stress range that causes a crack to develop is the stress actually at the site of the crack. Therefore, where there are stress concentrations due to changes in geometry, a stress concentration factor (SCF) must be applied to the general stress level.

For a plate under uniform stress, the stress intensity factor K may be estimated as:

$$K = \sigma \sqrt{\pi a F} \quad (2.11)$$

Values of SCF in the EUORCODE for plate cannot be found, weld connection value are available. In DNV there is a stress concentration factor for scallops that has significant dynamic pressure load on the plate. It depends on the geometry, because this is used to stiffen subject to axial load. [10] There are three approaches to determining the SCF:

- Experimental data
- Finite element analysis
- Parametric equation based on experimental data or finite element analysis

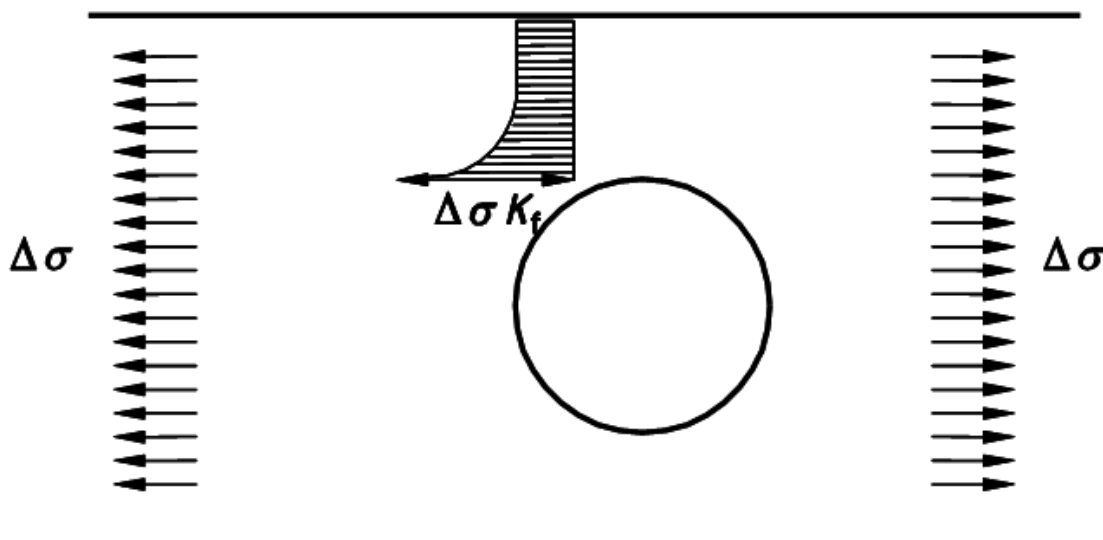


Figure 14 Example of stress concentration around a large hole [10]

The figure shows the stress concentration around bolt holes is by taking into account the category of the detail. Stress concentration effect can increase the stress locally, leading to local damage and crack initiation. The causes of stress concentration are sudden change in general geometry, loading of a member and local changes due to bolt and weld.

2.6.4. Stress range

The fatigue tests have stress difference by opposition to stress amplitude which is half this value, this is called stress range $\Delta\sigma$. It is the main parameter influencing the fatigue life of steel structural element. It defined:

$$\Delta\sigma = \sigma_{\max} - \sigma_{\min} \quad (2.12)$$

The fatigue life can be increased when part of the stress cycle is in compression. This is because of the residual stress; the behavior of a crack is in fact influenced by the summation of the applied stress and the residual stress.

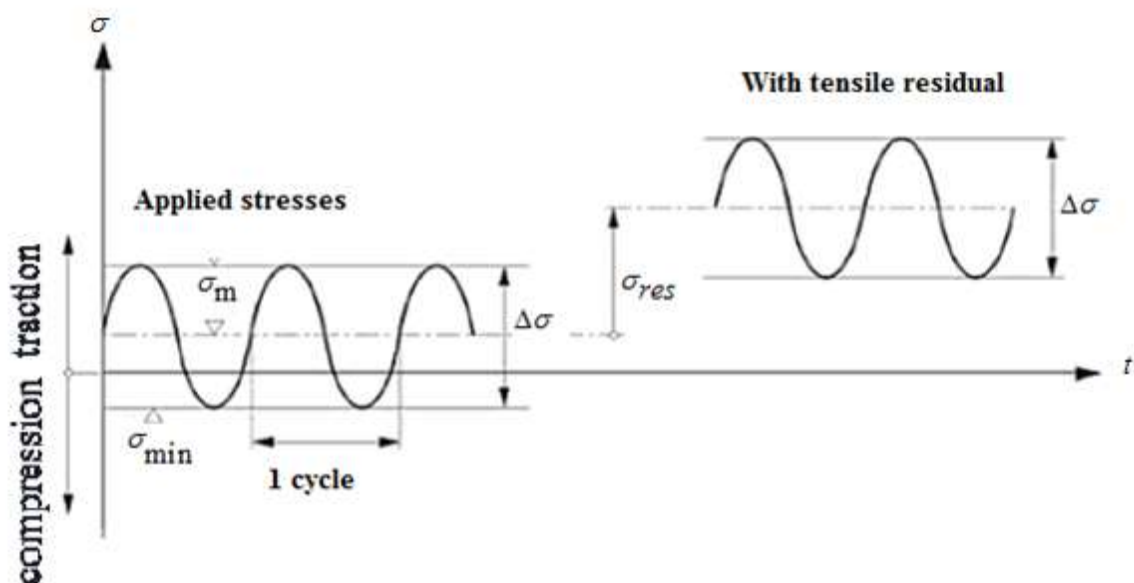


Figure 15 Illustration of a definition of stresses and influence of tensile residual stresses [\[1\]](#)

2.7. The theory of fatigue

Fatigue is a form of failure that occurs in structures subjected to dynamic and variable stresses. To understand what fatigue is, one must see why the beam breaks, can it break by pulling both ends or by bending it several time on the same location. The beam can break by bending repeatedly on the same point. First bend it down ward; bending stress induced in the beam, it will be tension on the top area and compression in down area. When the beam is in equilibrium there is no stress all over cross section. When it bends up ward there will be compression on the top and tension in the down cross section. This tells the reason behind fatigue, the fluctuating stresses on the material that gives micro crack. The crack growth then is brittle on the material. When a crack is initiated growing slowly due to dynamic load, even if the stress level is lower than the yield stress. The crack may gradually develop and become so long that the cross section reduction is too large. This causes the stave/structural elements breaking during applying a force, like a brittle material.

The geometry of the structure will have an impact on fatigue damage. A rectangular profile or sharp corners will lead to elevated local stresses where fatigue crack can start. Therefore that is important to use round and smooth transitions profiles for increase the fatigue strength of the structural details.

Fatigue crack can be provided on a jaw (a geometrical unexpected change) where the stress concentration is high. Typically a crack takes place on the welded connection and bolting. Both weld and bolt threads with leading stress concentration is damaging process is time-dependent and can span across time is often measured in the number of load cycles.

“In welded connection, starts fatigue cracking almost always on a weld defect, and grow period is more than 90% of fatigue life”. [\[23\]](#) In welded joints the crack initiation stage is unimportant. Because the weld defects attack always found in weld joints. Even if in perfect weld can contains defects along the weld connection or between weld locations.

The effects of stress at the weld toe are high; it is higher than the applied stresses. This is because of discontinuities in the welds. The cause of discontinuities in the welds can be several, which crack, imperfections, lack of fusion, etc. the result of those reduce the fatigue life that expected for a weld element.

Welding implies that steel materials locally are subjected to rapid melting and quickly cooling. This process causes structural changes, and large residual stresses in the heat affected zone (HAZ). A weldable must done without special safeguards or without higher risk of unwanted and dangerous irregularities in the weld. Therefore, it must meet the requirements for weldability as carbon equivalent, CEV. [\[11\]](#)

$$CEV = C + \frac{Mn}{6} + \frac{Cr+Mo+V}{5} + \frac{Ni+Cu}{15} \quad (2.13)$$

In addition a visual inspection of the whole length of all welds must be performed. The visual inspection shall be performed after welding is complete within an area, and before any other NDT tests performed.

“Fatigue criterion is an important criterion for all structures with dynamic repetitive loads. When man discussed the float criterion, come to the conclusion that was the average stress across large cross-sectional area that was essential and that local peaks at the notches (kjerver) were not so important. When fatigue is these local stress top the driving force for the injury, damage process starts as a highly local phenomenon but can be as time goes on increasing damage propagation so that we have a global impact with tear off of the entire cross section.”
[\[4\]](#)

Generally speaking, the structures life in terms of fatigue can be depend on the fact of geometry. It is mainly the material used or fabrication. The stress range, the structural detail geometry, the material characteristics and the environmental condition are the main parameters that influencing at the structures fatigue life.

2.8. The assumption of fatigue analysis

The effect of pitting corrosion on the fatigue life of structural steels used in bridge construction research work is the mean source for the necessary fatigue damage calculation. The research paper consists of same laboratory results that show; the loss of a plates thickness, nominal stress for the plates, pit depths and stress concentrations for instrumented corroded plates.

The load history on the bridge often shows an approximately that find by collecting data. It varies, traffic data show it can be having trucks that have weighing 355.8KN or it can be light car that weighing 8.9KN.[\[29\]](#) To get the relation between stress and load on the axis, can use the Weibull factor.

Fatigue failure is associated with the cumulative damage caused by repeated application of common levels of stress. Then one must design in terms of ultimate limit state that requires a different approach to design from other limit states. In general fatigue life depends on the actual stresses at positions that the shape of the member changes.

Many techniques can be used to analyze fatigue. Load-life, stress-life, strain-life and crack growth technique can be taking as example. All have advantages with some limitations. To see some;

Load-life assumption is load used to generate baseline data are the same as service loading. The advantage of this can be the actual test of structure, manufacturing and the local stress concentration. It has same limitation like mean stress effects cannot be included and new tests required for each change in material, load or geometry are to point out.

Stress-life major assumption is nominal stresses and material strength control fatigue life and accurate determination of K_f for each geometry and material can use.

The advantage of this, changes in material and geometry can easily be evaluated, large empirical database for steel with standard notch shapes. The limitation of mean stress effect is often in error and requires empirical K_f for good results.

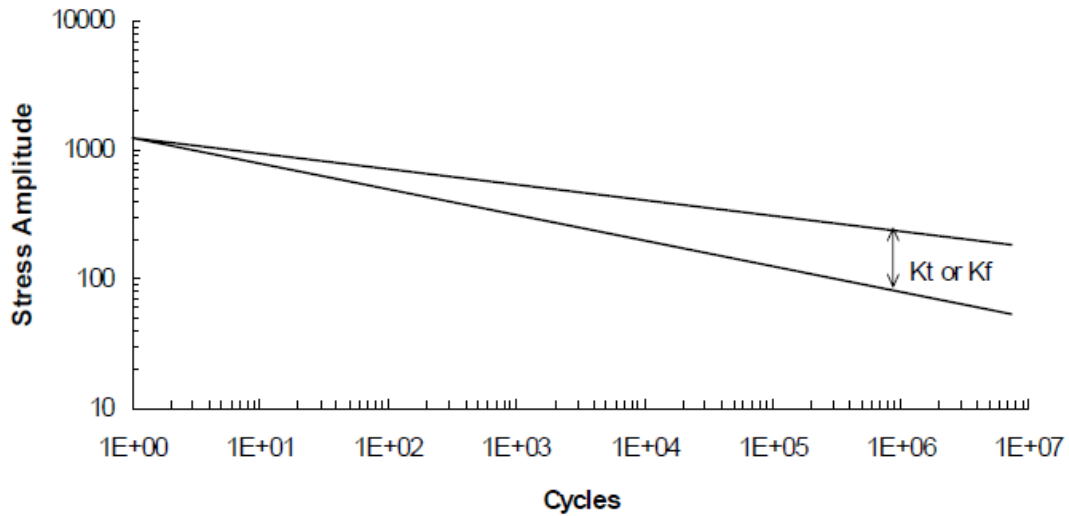


Figure 16 Notched S-N curve [28]

The log-linear relationship between stress on plate and a number of cycles at stress range can take approximately then it can correct with a Weibull factor [4].

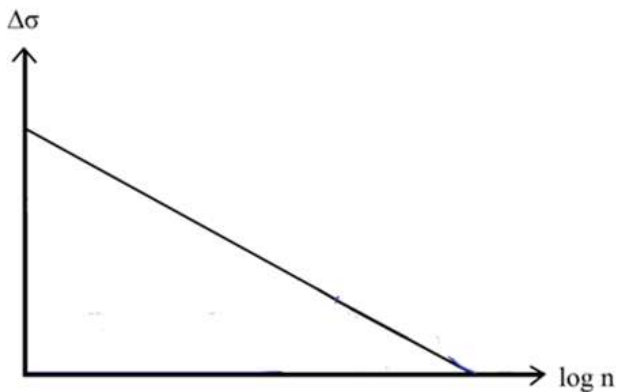


Figure 17 Illustration over linear relationship

The graph is then divided into several blocks such that the different stress and number of cycles for each load come into view [4].

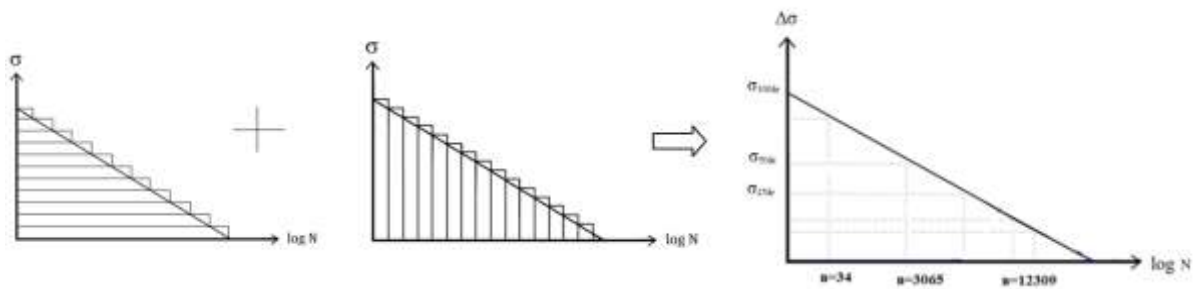


Figure 18 Illustration of the division of stresses and the number of cycles [4]

Registering stress range, which occurs around a crack growth over time, does calculation of fatigue in the first place. It is stress scorched ($\Delta\sigma$) that can be the driving force for damage process. Stress histories can be simplified by collecting equal stress scorched in groups, where each group is characterized by $\Delta\sigma_i$ and the number n_i .

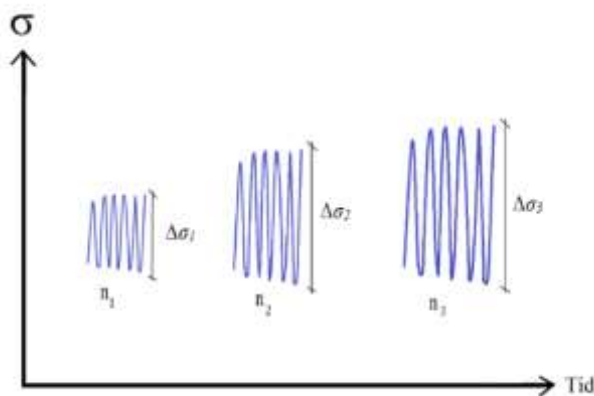


Figure 19 Stress range [1]

To calculate the total fatigue damage, all element damage must be collected. This is done for each bridge part will have its own damage.

The lifetime of a structural element determined by experiment provides important details. Choosing constant stress scorched can do this. Register number of alternations N to failure, it carries out each test at different stress scorched presents. The results will be graphically to every detail for a lifetime curve (S-N curve) curves given by the equation:

$$\log N = \log a - m \log \Delta\sigma$$

$\log a$ and M are constants for each line, M is the slope of the curve.

Fatigue criterion based on comparing the number of load cycles with stress-scorched n that

occurs in service (50 years) with a "safe" life N as determined by tests in the laboratory. Then be compared the number of performances load cycles during service time with lifetime N from the S-N curve, the criterion is defined as: $N > n$

2.8.1. Fatigue limit

Fatigue limit is the limit in maximum fluctuating stress a material can tolerate for an infinite number of cycles. It is usually determined from S-N diagram and is equal to the stress corresponding to the asymptote of the locus of points corresponding to the fatigue life of a number of fatigue test specimens.

Structures shall be designed to withstand the presupposed repetitive actions during the service life of the facility [1]. The design fatigue factors (DFFs) is given by divided in part in terms of classification of structural components based on damage consequence. The classification is also divided in to two parts, this one take in account the consequences of human life, significant pollution and major financial consequences.

2.8.2. Verification of fatigue

Verification of fatigue for nominal or geometric stress ranges due to frequent load $\psi_1 Q_k$ should not exceed: [12]

$$\Delta\sigma \leq 1.5f_y$$

Under fatigue load:

$$\frac{\gamma_{Ff} \Delta\sigma_{E,2}}{\Delta\sigma_C} \leq 1$$

$$\gamma_{Mf}$$

The fatigue assessment based on damage accumulation should meet the following criteria:

Based on damage accumulation:

$$D_d \leq 1$$

Based on stress range:

$$\gamma_{Ff} \Delta\sigma_{E,2} \leq \sqrt[m]{D_d} \frac{\Delta\sigma_C}{\gamma_{Mf}}$$

The load history is converted to a nominal stress history that is then applied to the component. Nominal stress is the engineering stress, which is the force divided by the original area.

According to DNV, nominal stress is understood to be a stress in a component that can be derived by classical theory such as beam theory.

The fatigue life can be calculated by using the S-N fatigue approach under the assumption of linear cumulative damage. As notes in DNV, the long-term stress range distribution is expressed by a stress histogram, consisting of convenient number of constant stress range blocks $\Delta\sigma_i$ each with a number of stress repetition n_i the fatigue criterion can be:

$$D = \sum_{i=1}^k \frac{n_i}{N_i} = \frac{1}{a} \sum_{i=1}^k n_i \cdot (\Delta\sigma_i)^m \leq \eta$$

Whereas:

D = accumulated fatigue damage

\bar{a} = intercept of the design S-N curve with the log N axis

m = negative inverse slope of the S-N curve

k = number of stress blocks

n_i = number of stress cycles in stress block i

N_i = number of cycles to failure at constant stress range $\Delta\sigma_i$

η = usage factor

= 1 / Design Fatigue Factor from OS-C101 Section 6 Fatigue Limit States

There are several alternative method of damage accumulated:

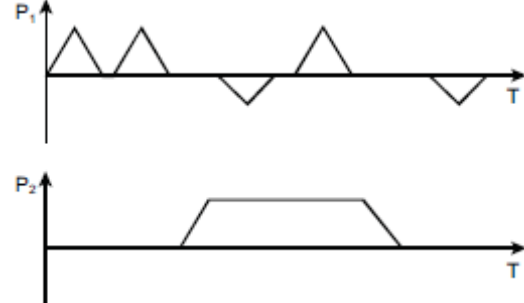

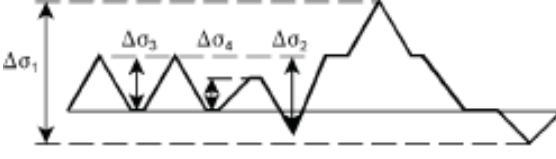
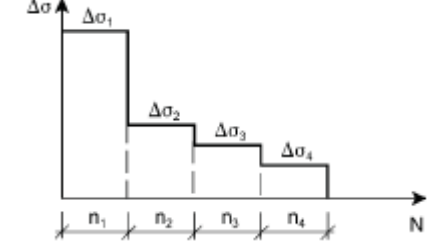
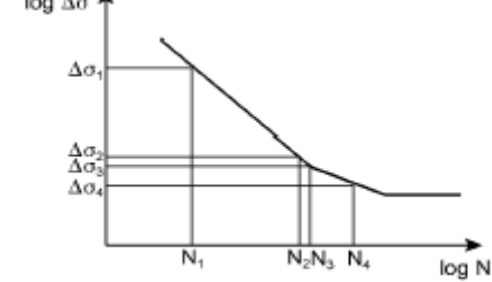
<p>a) Load sequence: Typical load cycle (repeated n-times in the design life)</p>	
<p>b) Stress history at detail</p>	
<p>c) Cycle counting (e.g. reservoir method)</p>	
<p>d) Stress range spectrum</p>	
<p>e) Cycles to failure</p>	
<p>f) Damage summation or Palmgren-Miner rule</p>	$\sum \frac{n_i}{N_i} = \frac{n_1}{N_1} + \frac{n_2}{N_2} + \frac{n_3}{N_3} + \frac{n_4}{N_4} \leq D_L$

Table 4 Cumulative damage method [12]

2.9. Corrosion fatigue

In the middle of 19th century many researches were published about corrosion fatigue behavior. Most of them were about the difference between alloys. The same researches concerns were expressed over the corrosion fatigue performance of certain alloys.

Corrosion fatigue and stress corrosion cracking has similarities, but there is significant difference in nature from corrosion fatigue and stress corrosion cracking. Corrosion fatigue crack propagation is absolutely time depending. Corrosion fatigue that time-cycle and cycle-dependent is complex and occur stress intensities. There are three basic types of corrosion fatigue:

- True corrosion fatigue (TCF)
- Stress corrosion fatigue (SCF)
- Mixed corrosion fatigue (TCF and SCF)

“True corrosion fatigue influences cyclic fracture even at maximum stress intensity factor K_{max} in fatigue less than K_{Isc} . Stress corrosion fatigue occurs only when K_{max} greater than K_{Isc} . [31] The material in an aggressive environment and subjected to repeated cyclic load that damage can be an example of corrosion fatigue failure. A corrosion fatigue failure for high-performance steel structures operating in deleterious environments is based on the likelihood of cyclically varying load and chemical environments in service. To get a predictable long life and life extension of steel structure one must use a steel material that can resist corrosion fatigue. This can be true by using qualitatively steel material, like duplex and super martensitic stainless steel.

When a material is subjected to cyclic loading, a localized structural damage takes place. This is progressive fatigue, which is caused by the local stresses. For example, steel prematurely subjected to repeated cyclic loading at lower stress levels, additional it is in aggressive environment, the result will be a corrosion fatigue fracture. The amount toleration depends on which type of steel was used.

Corrosion fatigue can be defined is fatigue in a corrosive environment. The aggressive

environment can be harmful for the fatigue life of a structure. Therefore corrosion protection is necessary and the design must consider corrosion in service, not only in view of fatigue. For a service life of a structure, corrosion is unwanted for reasons related to safety and economic uses. Corrosion can be avoided by considering the matter, which is selecting a good corrosion resistant material. The other alternative is applying a suitable surface protection, such as cadmium painting. Unfortunately, these options do not guarantee good fatigue properties. Most of high-strength materials have a relatively poor corrosion resistance. Accidental damage of steel structures has occurred due to fatigue cracks starting from corrosion damage, in several cases corrosion pits.

Corrosion fatigue should not be confused with stress corrosion, even if they have many similarities between them. Stress corrosion occurs along an intergranular crack (crack along the grain boundaries) growth path, whereas corrosion fatigue in many cases is still a transgranular crack (crack that follows the edges of lattices) growth phenomenon. Furthermore, stress corrosion does not occur in many technical materials, whereas corrosion fatigue can occur in most materials. It is not the same as fatigue of corroded materials. It is true corrosion damage can decrease the structural fatigue properties because it implies surface damage that could reduce the crack initiation life.

It is well known that a material subjected to a cyclic load far below the ultimate tensile stress can fail, a process called fatigue. If the metal is simultaneously exposed to a corrosive environment, the failure can take place at even lower loads and after a shorter time. Contrary to a pure mechanical fatigue, there is no fatigue limit load in corrosion-assisted fatigue.

The problem of corrosion fatigue for structure in service is characterized by variables that can be highly different from the material/environment system of laboratory research. A laboratory tests should concenter an acceptable time period whereas a structure in service is exposed to uncontrollable environment and exposure times of years. The effect of environment can be explained that a difference between fatigues of corroded specimens in a non-aggressive environment and fatigue of initially undamaged specimens in a corrosive environment. Crack initiation is possible at low stress amplitudes that would not have occurred without the corrosive environment. The environment can go into the crack after crack initiation. The involvement of corrosion can be chemical, electrochemical in liquid

environments or physical.

A time dependent process corrosion, can be the consequence of corrosion fatigue. Then corrosion fatigue should depend on the time scale of load history. Fatigue at a high frequency gives short life of structure. The reason is fatigue at a low frequency gives more time for a corrosion mechanism than fatigue at a high frequency.

The concepts of corrosion fatigue developed from stress corrosion cracking. The effects of corrosion fatigue are the combination of alloy, fatigue and environment. To understand corrosion fatigue, first step will be characterizing the behavior of those three main factors.

Corrosion fatigue is the acceleration of fatigue in an aggressive environment compared to the fatigue life in an unaggressive environment. Corrosion fatigue cracking can occur even environmentally, one must consider this when analyzing static loading, which mean cyclic loads where $K_{max} < K_{IEAC}$.

As shown on figure [10] a simple acceleration of the fatigue crack growth rate can be represented by an acceleration factor ϕ multiplied by the inert growth rate:

$$\frac{da}{dN_{aggressive}} = \phi \frac{da}{dN_{inert}}$$

The fatigue threshold ΔK_{th} for the inert environment can be calculated by using the above equations. That is normal a cycle-dependent corrosion fatigue occurs in environments that do not result in significant environmentally assisted cracking (EAC) under static loading. Where the mass transport and electrochemical reaction that contribute to fatigue acceleration are happening in a very short time.

2.9.1. Background

[Fricke] In 1971 realized that the characteristics of corrosion fatigue crack growth behavior were distinctly different above and below the threshold stress intensity (K_{ISCC}) for stress corrosion. These two types have since been named as true corrosion fatigue for stress intensities below K_{ISCC} and stress corrosion fatigue for those above K_{ISCC} .

The appearance of corrosion fatigue behavior divided into three basic types. Those are:

- True corrosion fatigue (TCF)
- Stress corrosion fatigue (SCF)
- Combination of TCF and SCF

(a) True corrosion fatigue that conjoin fatigue cycling and environmental aggression is responsible for enhanced rate of crack growth and can occur throughout the range of stress intensity. This is cycle-dependent corrosion fatigue behavior.

(b) Stress corrosion fatigue on the other hand of TCF, occurs only when the stress intensity exceeds the threshold for stress corrosion. This is time-dependent corrosion fatigue behavior.

(c) Combination of this two, thus true corrosion fatigue applies to materials which do not stress corrodes whereas stress corrosion fatigue is simply the result of stress corrosion under cyclic loading. For the situation of materials, which suffer stress corrosion and also exhibit true corrosion fatigue, a situation that is therefore a combination of stress corrosion fatigue and true corrosion fatigue. This is cycle-dependent and time-dependent corrosion fatigue behavior in somebody's company.

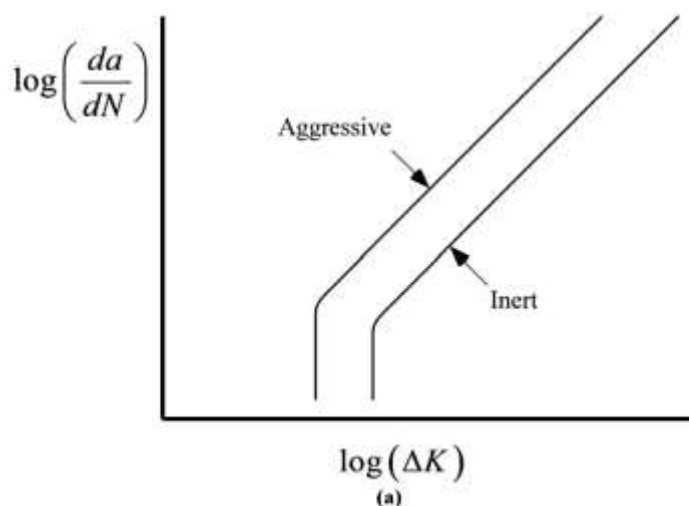


Figure 20 Cycle-dependent corrosion fatigue behavior [3]

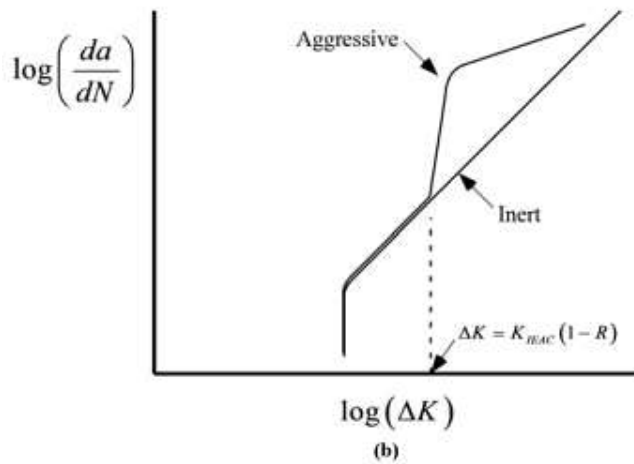


Figure 21 Time-dependent corrosion fatigue behavior [3]

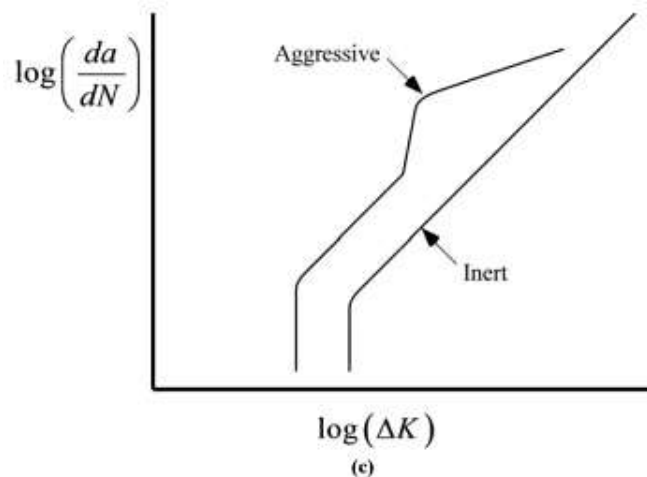


Figure 22 Cycle-dependent and time-dependent corrosion fatigue behavior in somebody's company [3]

Figures show the appearance of the three basic types of corrosion fatigue behavior.

To model the time-dependent corrosion fatigue[] can be use a simple superposition of the inert fatigue crack growth rate with the environment cracking rate.

$$\left(\frac{da}{dN}\right)_{\text{aggressive}} = \left(\frac{da}{dN}\right)_{\text{inert}} + \frac{1}{f} \left(\frac{d\bar{a}}{dt}\right)_{\text{EAC}}$$

Most of construction steel material has the same behavior on environment combinations.

Those display both, cycle-dependent behavior and time-dependent behavior

By taking in account the combination of the those two equations above [] can be get a more general expression for corrosion fatigue:

$$\left(\frac{da}{dN}\right)_{aggressive} = \phi \left(\frac{da}{dN}\right)_{inert} + \frac{1}{f} \left(\frac{d\bar{a}}{dt}\right)_{EAC}$$

At different frequency' levels the crack growth rate will be different. The crack growth rate approaches the inert rate at high frequencies. This is because the environmentally assisted cracking growth per cycle is negligible. The environment crack growth per cycle dominates over fatigue and the rate is proportional to $\frac{1}{f}$ at low frequencies.

CHAPTER 3 PROPOSED FATIGUE MODEL FOR PITTING CORROSION

3.1. Effects of corrosion on fatigue of steel

The general corrosion of steel plates used in various bridge components results in surface roughening and pitting. Oxygen deprivation typically occurs when plates are bolted, riveted or are butted up against each other, this kind of attack called crevice corrosion. Most of load-carrying steel structural components are affected by corrosion, which are included plate's girders, floor beams and plates and gussets.

The fatigue strength of material before corrosion is that stress level below which the material can tolerate an infinite number of stress cycles without failure. The material alloy steels and carbon steel in air has the fatigue strength approximately half of tensile strength. When the material corroded this proportion no longer holds, and fatigue strength is seriously degraded at high cycle fatigue. [\[29\]](#)

3.2. Effects of corrosion on the fatigue failure equation

The fatigue strength of structural steel significantly affected by general corrosion, pitting corrosion and formation of deep crevices corrosion. In various hot rolled structural steel that containing substantial alloy additions of copper, sulfur, phosphorus and chromium, the formation of deep, sharp pits can cause a severe reduction of fatigue strength at 10^7 cycles from 23ksi (158Mpa) when corroded, according to the studies of Albecht, et al. [\[29\]](#)

The equation of fatigue failure illustrates by using three variables that are affected by corrosion:

$$N = C \cdot [S]^m \quad (3.1)$$

- Exponent (m)
- Coefficient (C)
- Stress range, ksi (S)

The mechanical forces of crack propagation within steel have primarily relation with exponent (m), with the corrosive environment causing small changes in the mean slope of -3.26 ± 0.50 . The estimated slope of these largely parallel lines of the various fatigue categories in $m = -3.26$.

For fatigue category A or B fatigue strength changes as a function of time then the surface conditions are being modified by corrosion. Contrasting laboratory fatigue studies where the ground or hot rolled surface remains intact, gradual changes induced by corrosion take place over the course of many years to change fatigue strength in a bridge. The ambient stress state also increases as sectional losses are sustained, assuming that present loading remain the same. Traffic loadings may progressively increase often.

A general corrosion fatigue equation is proposed, taking general corrosion and pitting into account:

$$N = \left(\frac{C}{K_f} \right) \cdot [S_{corr}]^m \quad (3.2)$$

Where: N is number of available fatigue cycles at a given stress level

C is fatigue strength coefficient

S_{corr} is stress range in member, adjusted for moment of inertia or section loss due to Corrosion

K_f is fatigue reduction factor, related to pitting for structural steels

The term S_{corr} is unnecessary if strain gages are used for the measurement of stresses in structural steels in the corroded critical sections. Because, the strain gage reflect actual stress. However, may be use of stress concentration factors can be necessary, because it is difficult of placing gages on corroded sections.

In this investigation, the effects of surface attack, pitting penetration and section size were determined for actual corroded steels. This included the verification of the fatigue reduction factor K_f and the effects of section loss on the term $[S_{corr}]$. [\[29\]](#)

3.3 Stress concentration and fatigue strength

A structural member sustaining both surface pitting and section loss is subject to a phenomenon termed stress concentration due to significant geometrical changes. Surface pitting decreases the initiation time for fatigue cracks to form. Penetration by corrosion influences both local and overall gross stresses in structural members. Unfortunately, stress concentration effects are not easily divided by a sharp demarcation of surface pitting influences vs. predictable geometrical changes because corrosion itself is not entirely uniform.

There may be sharp pits in one section, whereas there might be gradual section changes with shallow pits in an adjacent area of the structural member.

To verify the effects of pitting and section change on stress concentration, strain gage were actually mounted on corroded section cut from steel plates that were removed from several older bridges. The plates were cut into coupon shapes and a 7/8" diameter hole was drilled on each and for load application in a universal tensile testing machine. The profiles of each specimen were measured with micrometer Vernier calipers and pit depth indicators accurate to 0.001" (25.4 micrometers). The specimens varied in length, and were loaded to obtain a nominal stress of 1,000psi (6.895Mpa) in their original cross sectional area. The general geometries of the some of specimens are shown in figure 23 through 26.

After characterizing each specimen, a strain gage was mounted on the specimens in order to compare calculation nominal strain based on net section with actual strains where pits or section loss occurred. The strain was measured with foil strain gage at various locations on the specimen, particularly at pitted or penetrated areas, resulting in a stress concentration factor defined as follows:

$$K_f = \frac{\text{actual stress in corroded area}}{\text{nominal stress in corroded net section}} \quad (3.3)$$

These tests results can see from the table on the same chapter. It compares pit depth vs. K_f and original specimen thickness divided by pit radius vs. K_f . Intergraph computer-aided drafting (CAD) systems were determined the pit radius.

The fatigue reduction factor K_f as a function of pit depth data exhibits wide scatter as shown in the next chapter. [63] It has a fair Pearson correlation coefficient of $r=+0.719$ and it is roughly linear. Pit depths vary widely in a specimen then expected a lower value of correlation coefficient. In this investigation, steels tested either conform to ASTM A36 or ASTM A7 (an obsolete specification). The fatigue reduction factor lines of Albrecht et al. are drowning for comparison. These lines represent plain carbon and weathering steel that show weathering steels having the severest reduction and plain carbon steels having the least reduction. In this investigation, the best-fit line of the strain gage data is lies right in-between

the weathering and plain carbon steel lines of Albrecht and Co-workers. The best-fit line for strain gage will evaluate:

$$K_f = 1.2 + 5.77 \cdot P \quad (3.4)$$

Whereas:

P is pit depth, inches

K_f is stress concentration

Insufficiency of using pit depth alone as a predictor of strain concentration in that it does not take section change, pit radius and size effects into account. The theory of notches and grooves of plates in bending and tension have extensively studied in the literature and are well summarized by Peterson. [\[29\]](#)

CHAPTER 4 VARIFICATION OF PROPOSED MODEL

4.1. General

This section covers steel bridge member that the critical members/plates simulated by the structural program SAP2000 in terms of corroded area.

The investigation of the actual geometrical measurement, reduction of the steel plate thickness due to corrosion and all effective parameters that must done by laboratory material are handle by the research group [\[29\]](#).

The material properties is very important for all structural members, therefore one should know the property of material used for steel bridges for assess the remaining fatigue life of the bridge member. The stress on the material due to combination of load and corrosion depend on the material property and strength. For Illinois Bridge are used ASTM A36, A588 and A517.

To evaluate the corrosion state one can use the visual inspection. If the bridge members are corroded, the remaining thickness of the particular cross section has to be measured and considered for remaining static checks.

The steel bridge structural plates are modeling by computer program SAP2000 to determine action effect of plate thickness. The modeling of plate is designed as single thin plate in 3D. It helps to identify the behavior of the plates, in 3D modeling like, the natural axis of the plate, deflection, stress in different component of stress, etc. all depends on the plates in which position are taken to see the plates behavior.

A verification of steel bridge structural element in terms of fatigue life is very important for all old bridge construction. Strength of all structural steel depends on the environmental condition, the type of load and period. The bridge elements can reduce the thickness uniformly or partially. The main cause can be corrosion, during the life time of a structure it can be subjected to different failure mechanisms. Verification of remaining fatigue life of corroded bridge members is the main focus in this thesis. Steel plates which are used as a bridge elements are subjected to pitting corrosion. Because of this the plates reduce the

thickness irregularly. The plates be modeled and analyzed by using the computer program SAP2000. The main purpose of this study is to verified the remaining fatigue life of the corroded plates by using S-N curve method.

Modulation of the steel bridge element and analization of it has been done by using the program that is called SAP2000. [\[A.1\]](#) The program can calculates and analyze the structures consisting of beam, plate and 3D elements using the Finite element method.

4.2. Steel bridge structural elements verification

The verification of steel bridge structural elements based on standards that gives proper requirement. Fatigue can by verified by using analysis of details with stress analysis that shows on the above section [\[2.8.1\]](#). The nominal stress concept to assess the fatigue safety that the design nominal stress range that come from the fatigue action effects, can compared to the design value of the fatigue strength. There are three formats:

- Verification using the fatigue limit.
- Verification using damage equivalent factors.
- Verification using damage accumulation.

Using the fatigue limit, fatigue test on specific details can be carried out. Fatigue test can be done by taking a sample element from existing structure or use an element that fabricated especially for testing purpose. Using an element fabricated especially for testing purposes, could represent possible existing details in terms of geometry an material capacity. For structural steels, fatigue tests under variable amplitude stress ranges show that the life of a structural detail tends to be infinite if all design values of the stress amplitudes remain below the calculated value of fatigue strength.

According to fatigue limit acceptance, the total number of cycles for stress histogram one can assumed $5 \cdot 10^6$ cycles.

Using damage equivalent factors based on a modified nominal stress range approach or geometric stress approach. The design value of the equivalent constant amplitude nominal stress range can be at $2 \cdot 10^6$ cycles. [\[10\]](#) The verification condition can be:

$$\lambda_{\max} \cdot \Delta\sigma(\gamma_{Ff} Q_k) \leq \frac{\Delta\sigma_c}{\gamma_{Mf}} \quad (4.1)$$

$$\text{Whrer as } \gamma_{Ff} \Delta\sigma_{E,2} = \lambda_{\max} \cdot \Delta\sigma(\gamma_{Ff} Q_k)$$

$$\therefore \gamma_{Ff} \Delta\sigma_{E,2} \leq \frac{\Delta\sigma_c}{\gamma_{Mf}}$$

Using damage accumulation method determine origin of the damage provides the stress ranges, generally the values below the fatigue cutoff limit neglected. The limit value for the damage sum is indicated as $D_{\max} = 1$ [12]. The verification format is:

$$D_d = \sum D_i = \sum \frac{n_{Ei}}{N_{Ri}} \leq D_{\max} \quad (4.2)$$

According to Eurocode 3 [6], verification of plate cross section resistance in ultimate limit states should be considered. Plate with cross section for uniaxial bending should be performed:

$$\eta_1 = \frac{N_{Ed}}{f_y \cdot A_{eff}} + \frac{M_{Ed} + N_{Ed} e_N}{f_y \cdot W_{eff}} \leq 1 \quad (4.3)$$

$\gamma_{M_0} \quad \gamma_{M_0}$

The partial factor γ_{M_0} recommended value is one in all standards [26]. If it is relevant the internal forces N_{Ed} and M_{Ed} second order effects included. For members subject to compression and biaxial bending the above equation may be modified as follows: [6]

$$\eta_1 = \frac{N_{Ed}}{f_y \cdot A_{eff}} + \frac{M_{y,Ed} + N_{y,Ed} e_{y,N}}{f_y \cdot W_{y,eff}} + \frac{M_{z,Ed} + N_{z,Ed} e_{z,N}}{f_y \cdot W_{z,eff}} \leq 1 \quad (4.4)$$

$\gamma_{M_0} \quad \gamma_{M_0} \quad \gamma_{M_0}$

4.3. Basis of simulation

There is a higher probability of failure due to fatigue and corrosion for most of old steel bridges. The static overloading has impact for structural failure but it is not the mean cause of fracture. According to data from the report "Physical research report No. 108" the fatigue life of Illinois bridge determined to be 86 years.

The verified elements are steel flat plat that corresponds in the detail category 160 it is the same as B1 on S_N curve.[9] On the basis of the data on the relevant cross section, the stress ranges and the number of crossing when obtained. The research result is the base of the

simulation of steel bridge members in this paper it has the result of the plate thickness reduction and the depth of pit. Then finding stress after pit is using the program SAP2000.

4.4. Control of steel bridges structural critical plates

Stress on the plate can find by using the principle of mechanics of solid. For this paper, use the result from the program "SAP2000". Fatigue calculated by used the highest stress on the system.

Fatigue cracks can occur from the localized corrosion then plate degrader in local region. The corrosion damage of steel bridge member (plate) can influenced by many factors, such salt on the winter season, water, virus and loss of the corrosion protection, because of age or friction with vehicles.

Generally the distribution of pit corrosion on a plate can going in different directions. For this thesis, the maximum distance of localized corrosion is 152.393mm. The maximum depth of pit corrosion is 6.223mm.

4.5.1. Stress concentration factor

Stress concentrations are highly localized effects, which are functions of geometry and loading. The other name of stress concentration is structural discontinuities. These structural discontinuities can be all types of geometrical change such as holes, notches and retardation of plates.

On the side of tall about the actual stress resulting from a geometric stress raiser, a theoretical stress concentration factor is applied to the nominal stress. For a plate subjected to a normal stress, the actual stress in the geometric discontinuity can be:

$$\sigma_{\max} = K_t \sigma_0 \quad (4.5)$$

Whereas:

K_t is theoretical stress-concentration factor

σ_0 is nominal normal stress

The definition of the stress concentration factors is the maximum principal stress divided by the nominal stress. For this paper, the maximum principal stresses are from FEM (SAP2000) results; fatigue reduction factor/stress concentration factor K_f is calculated:

$$K_f = \frac{\sigma_e}{S} \quad (4.6)$$

4.5. Dimension of the chosen plate

The investigation of the actual geometrical, reduction of the steel plate thickness due to corrosion and all effective parameters that must done by laboratory material are handled by the research group. [29]

The material properties is very important, one should know the most important material characteristics to assess the stress in the element and to asses remaining fatigue life of the element. For Illinois Bridge ASTM A36, A588 and A517 was used.

The plates that modeled by SAP2000 and the name of specimen will demonstrate in this section. If one want see more details could read the report's [29].

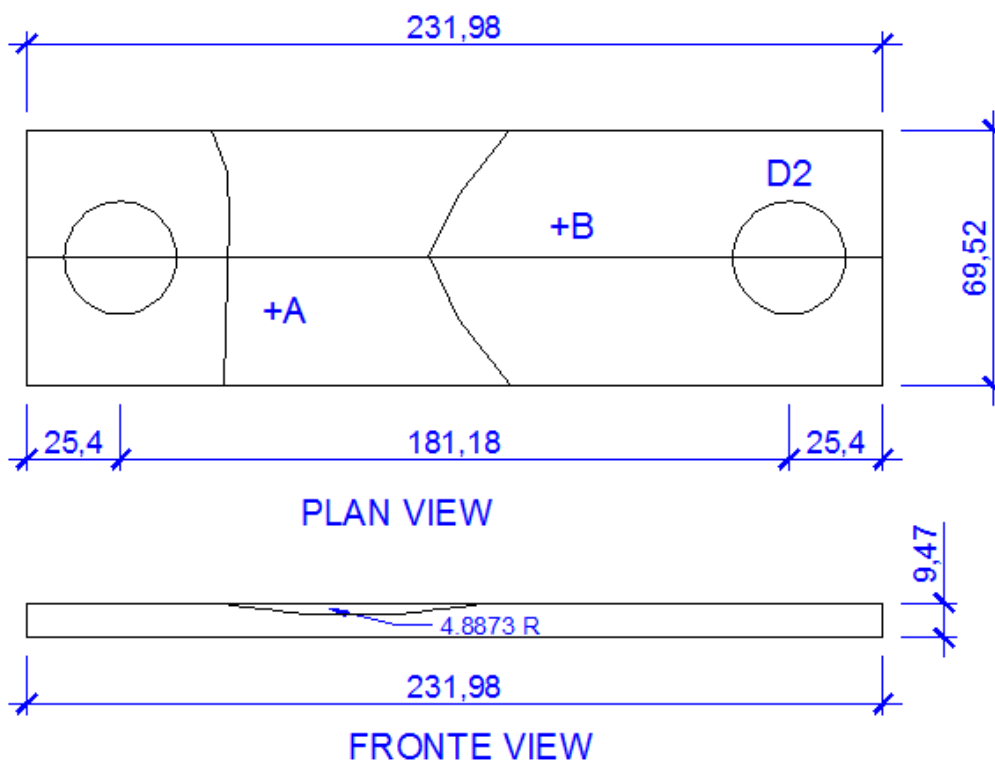


Figure 23 Dimensions and pitting in specimen D2 for SAP2000 model [58]

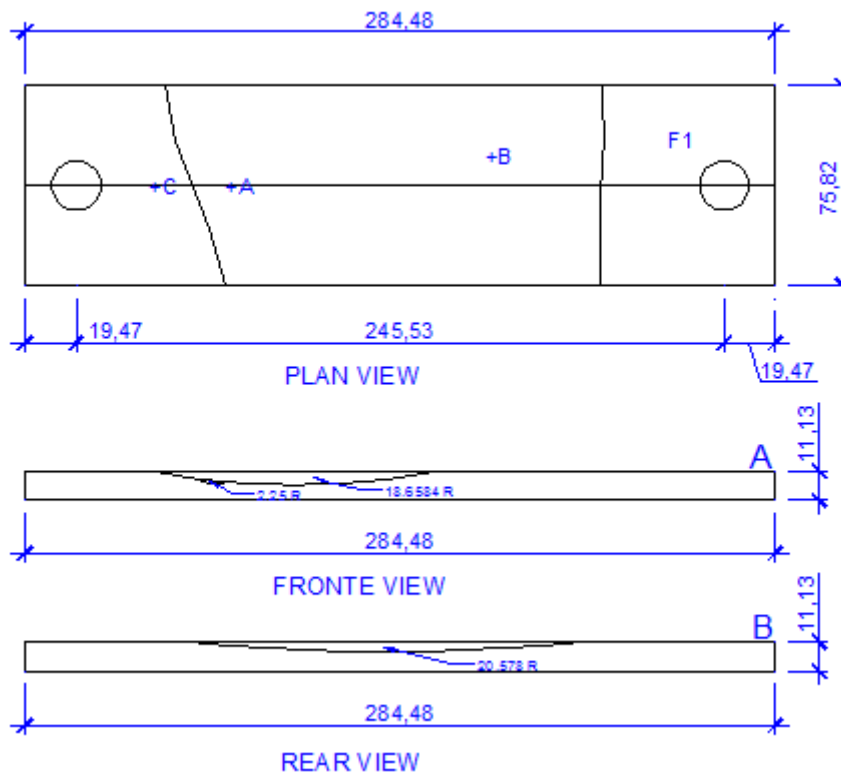


Figure 24 Dimension and pitting in specimen F1 for SAP2000 model [59]

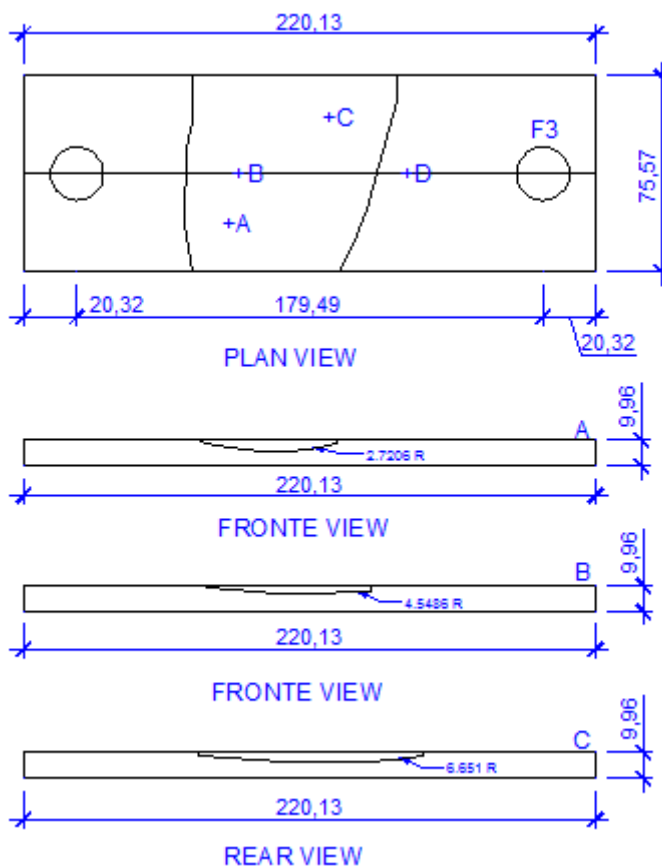


Figure 25 Dimensions and pitting in specimen F3 for SAP2000 model [60]

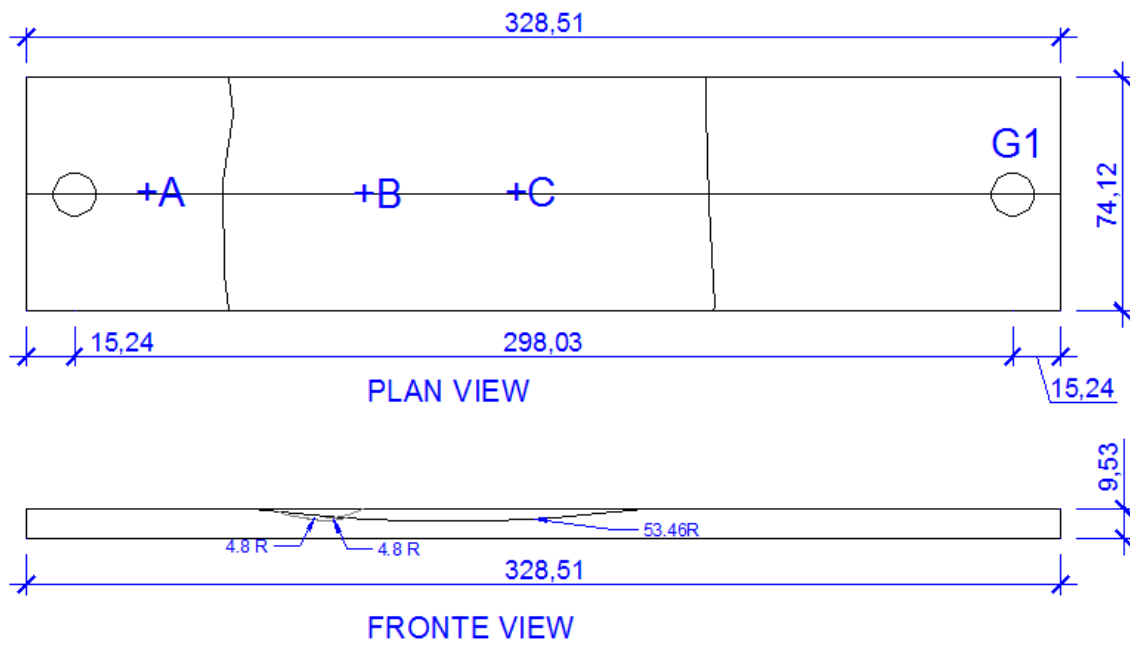


Figure 26 Dimensions and pitting in specimen G1 F3 for SAP2000 model [61]

4.6. Actual stress

The actual stresses that used for to find the stress concentration [14] factor are fund from simulation on the program [A.1]. In this section, show the value of actual stresses simulation in figure form.

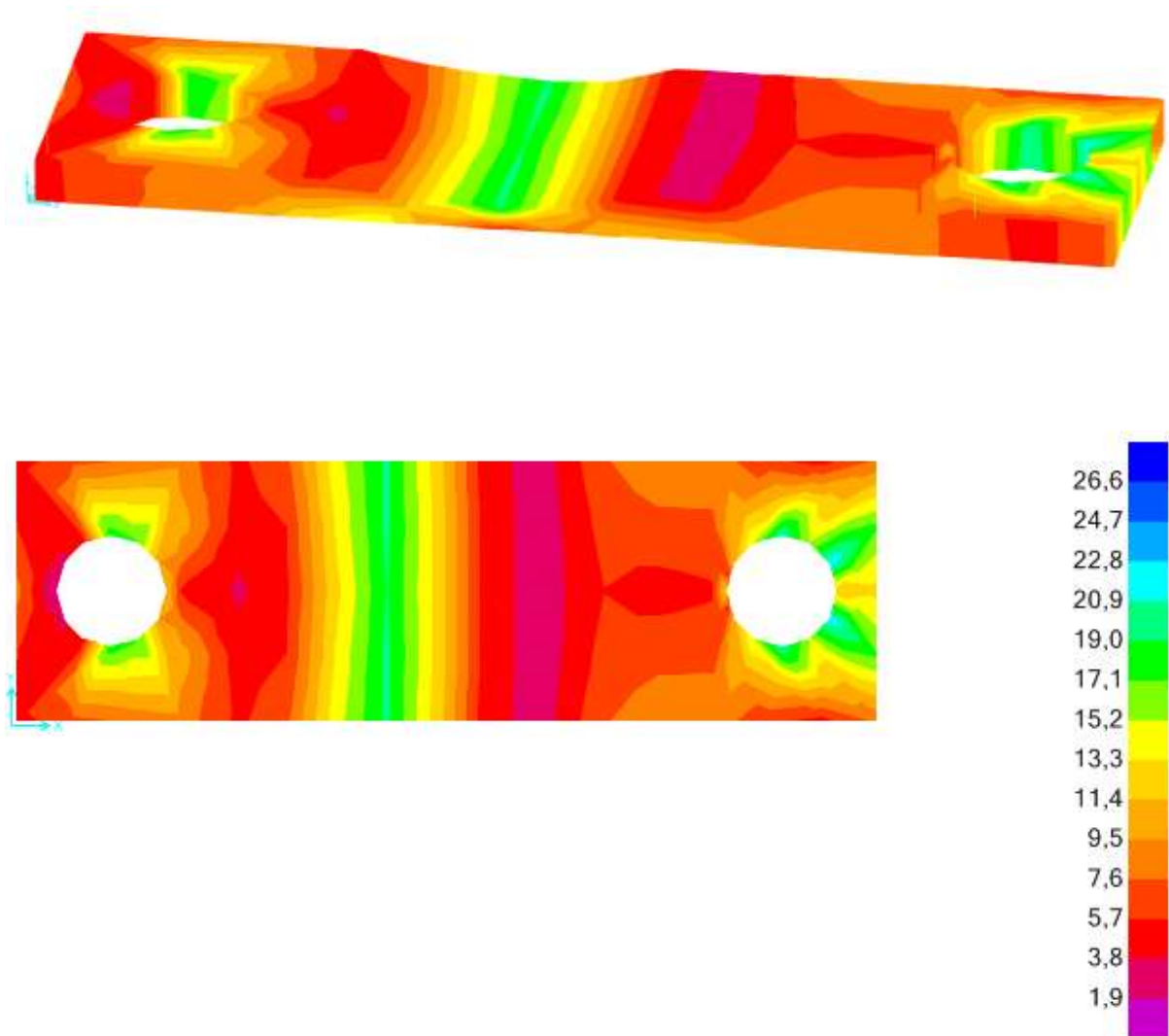


Figure 27 Actual stress for plate 1

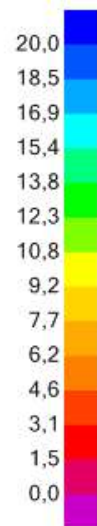
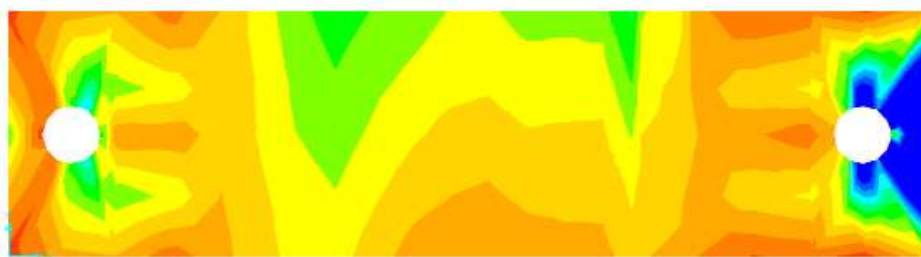
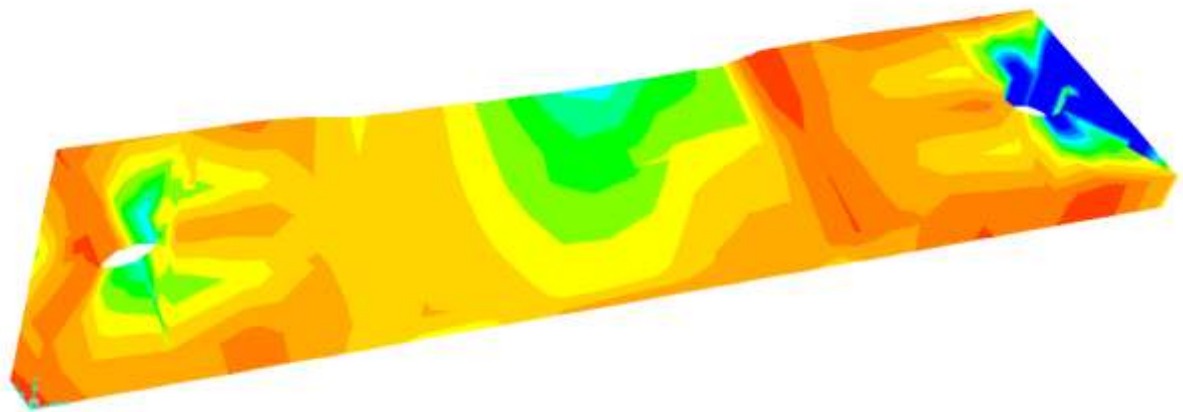


Figure 28 Actual stress for plate 2

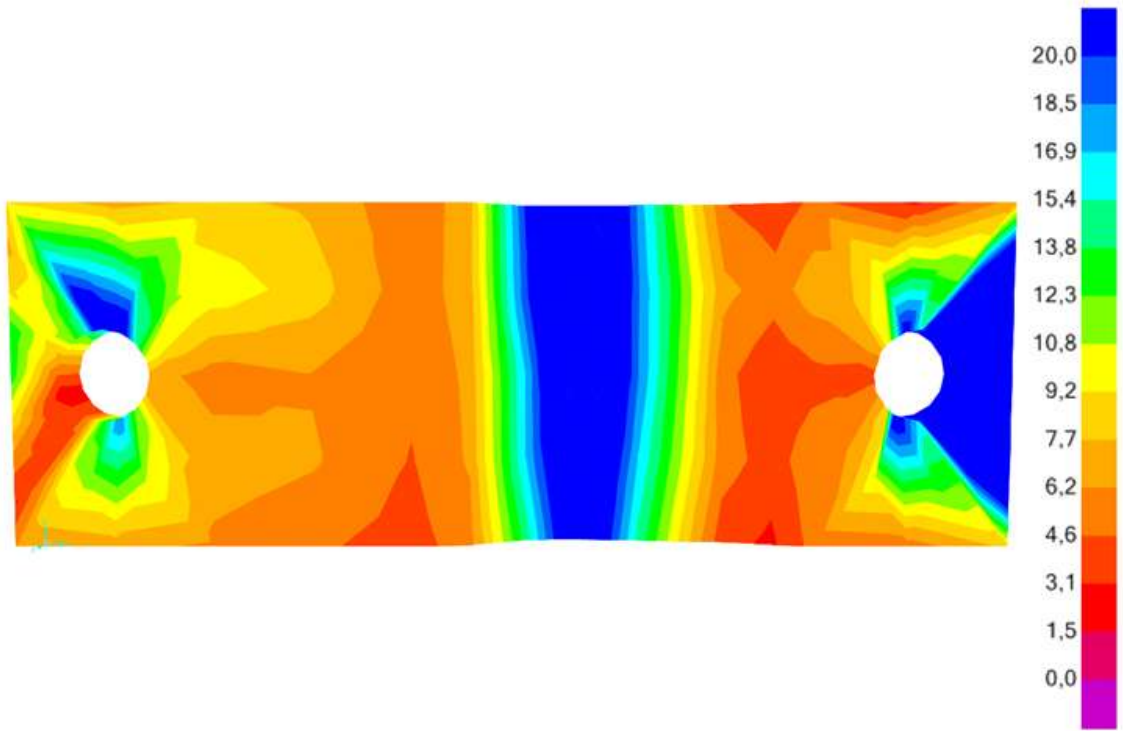
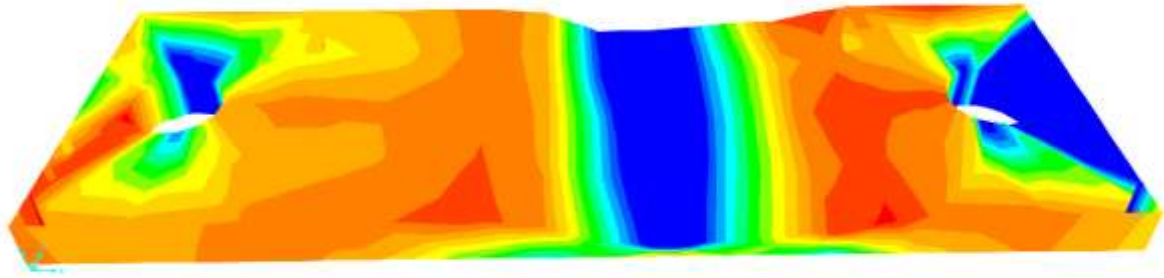


Figure 29 Actual stress for plate 3

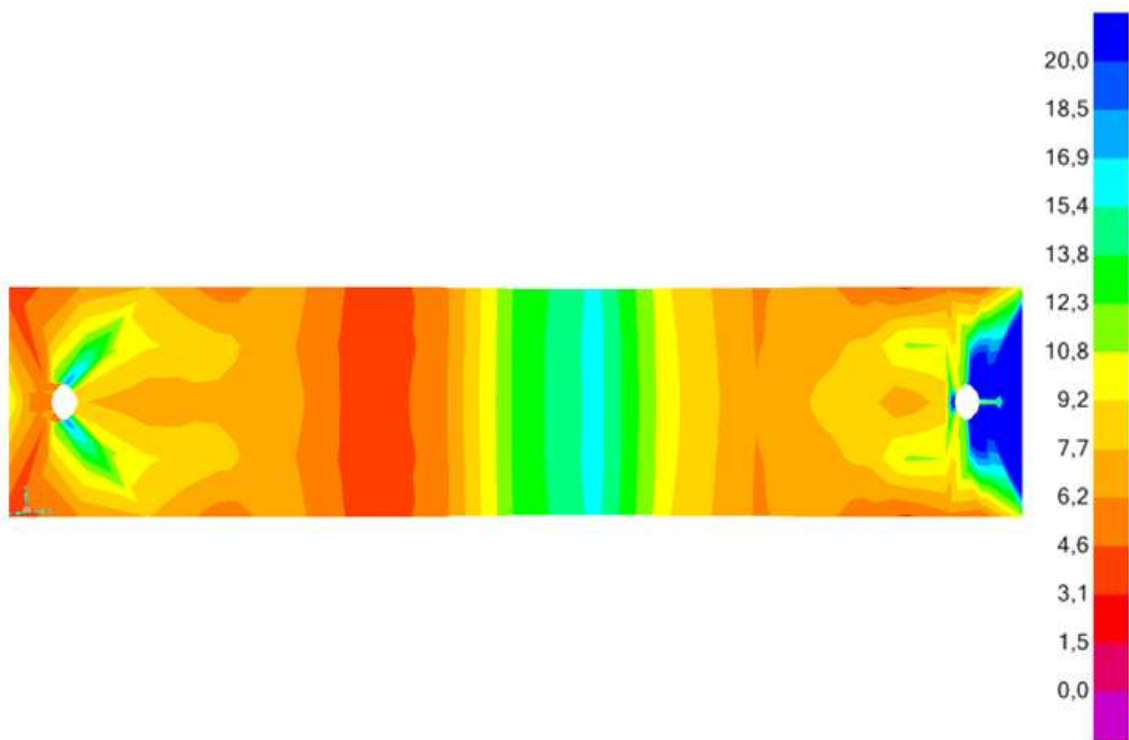
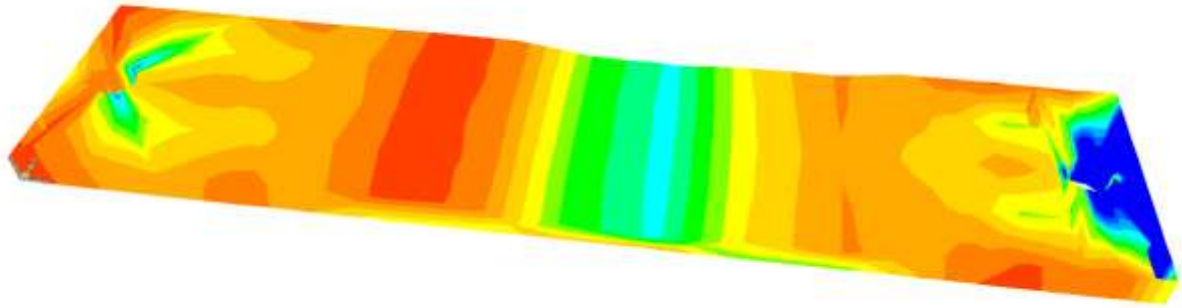


Figure 30 Actual stress for plate 4

4.7. Stress concentration factor

Fatigue cracks most commonly initiate from points of stress concentrations in structural details. In this section one can see how stress concentration factor graph been plotted by using strain gage data: [29]

Pitting corrosion is considered to be one of the principal degradation mechanisms for many metallic materials subjected to corrosive environment. The fatigue life was shortened due to formation of corrosion pits on surface of the material causing the initial damage and then cracks initiated from these pits

The stress concentration effect of different corrosion pits is different. Corrosion pit size varies as exposure time in solution increases and depends on electrochemical and mechanical conditions. The shapes of corrosion pit can be seen as semi-circle. The stress distribution at the semi-circle corrosion pits is the main parameter affecting the stress concentration factor (SCF). Usually used in structural design the effects of equivalent stress concentration factors to analyze the effects of pitting corrosion on fatigue life.

pit or penetration depth (d) inc.	Specimen	Radius (R) Inc.	Thickness (M) inc.	$\frac{M}{R}$	$\frac{dM}{R}$	K_f
0.245	G_1	4.2	0.367	0.093	0.0228	2.60
0.158	G_3	3.92	0.375	0.096	0.0152	1.59
0.100	G_3	52.0	0.375	0.007	0.0007	1.41
0.119	F_3	6.7	0.373	0.056	0.0067	2.13
0.032	F_2	134.0	0.382	0.003	0.000096	1.33
0.042	F_2	21.4	0.382	0.018	0.00076	1.46
0.188	F_1	2.25	0.438	0.195	0.0367	3.22
0.193	F_1	20.5	0.438	0.021	0.0041	2.18
0.175	G_2	53.5	0.375	0.007	0.00123	1.73
0.099	D_2	4.9	0.375	0.007	0.00762	2.33
0.203	D_2	4.9	0.375	0.007	0.0156	2.78
0.242	F_3	2.7	0.392	0.145	0.0351	2.37
0.231	F_3	4.5	0.392	0.087	0.0201	2.07

Figure 31 Pit depths, radius, specimen thickness and stress concentrations for instrumented corroded plates [29]

Figure	Specimen	Pit depth (d) (inch)	Max. Von Mesis Stress σ_e (Mpa)	Nominal stress S (Mpa)	K_f (FEM)	K_f (Experimental)
42	D_2	0.203	19.00	6.859	2.77	2.78
	D_2	0.099	–	–	–	2.33
43	F_1	0.193	15.65	6.859	2.28	2.18
44	F_3	0.240	20.77	6.859	3.03	2.37
	F_3	0.231	16.40	6.859	2.39	2.07
45	G_1	0.245	15.99	6.859	2.33	2.60

Table 5 Pit depth, Von Mises & nominal stresses and stress concentration factor for corroded plates

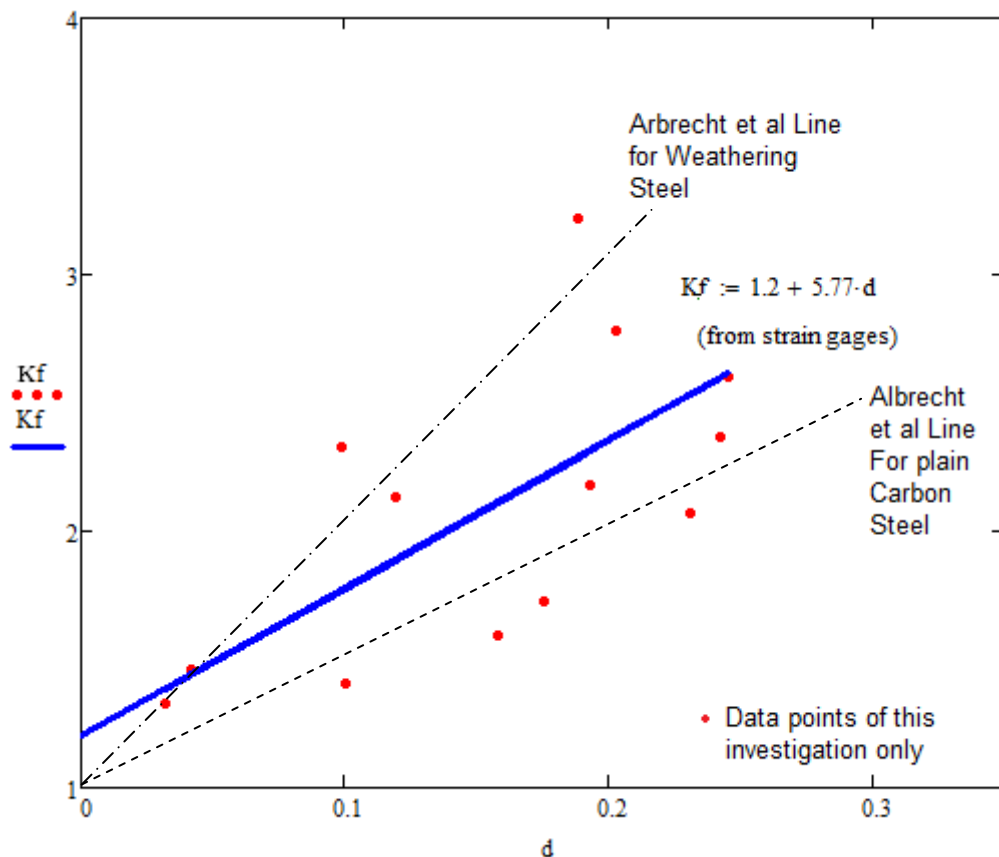


Figure 32 Stress concentrations derived from strain gage data plotted vs. the pit depths in the plate specimens [29]

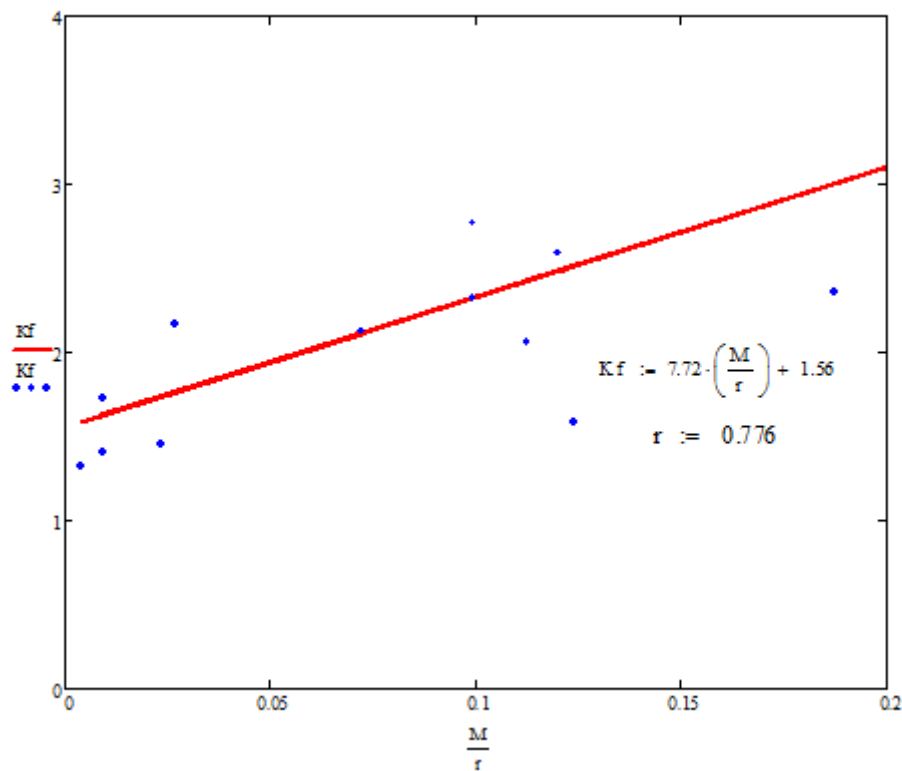


Figure 33 Stress concentration factor K_f in corroded plate specimen was derived from strain gage data and plotted vs. the function of plate thickness derived by pit radius. The correlation coefficient, r , for this data plot is +0.776 [29]

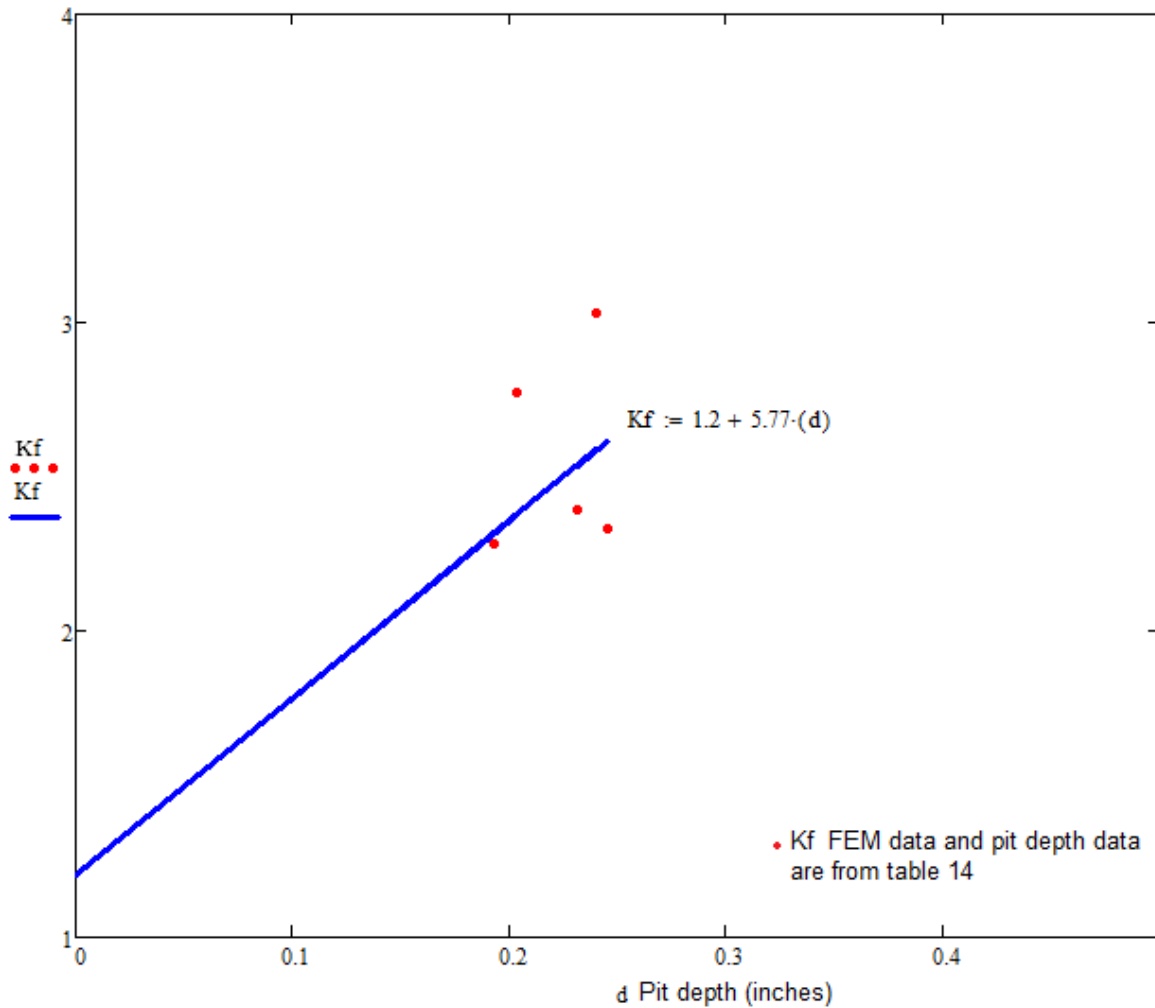


Figure 34 Stress concentration derived from FEM plotted vs. the pit depths

Conclusion

These results of stress concentration factor that were derived by both techniques are almost the same. Then one can use these stress concentration factor for analysis how concentrated the stress in a bridge element.

CHAPTER 5 CASE STUDY

5.1. Steel member fatigue analysis method

The analysis of fatigue is based on S-N curve, the stresses can be calculated in agreement with the definition of the stresses to be used together with a particular S-N curve. According to Norsok [25] the aim of fatigue design is to ensure that the structure has a sufficient fatigue life. The structural design fatigue life must be specified by the operator in terms of the structure service life; if it is not specified, a service life of a structure design could be limited to 15 years.

Existing structure must have a specified residual service life, the reliability assessments aim is to produce the evidence safety. The reliability of steel bridge structural elements designed for fatigue decreases with time in service, the reason is the steel bridge structural element can be subject for repetitive load and is ongoing damage on the elements.

Structural engineers propose different strategies to evaluating the fatigue reliability. The goal of this study is to fetch the structural safety. The study takes in account the consequence of the safety class, in what level is the risk of accepting low safety class. For example, the study assumed fatigue strength limit, assumed no fatigue crack during the whole service life and assumed the structure can tolerate extensive fatigue cracking without failing.

5.2. Steel element fatigue strength analysis method

The assessment of the remaining fatigue life of an old steel bridge structure, critical structural members have to be identified and categorized. To categorize the quantitative relationship between the stress ranges, there are the algebraic differences between the two extremes of a particular stress cycle derived from stress history. Categories structural details are described in Eurocode [6].

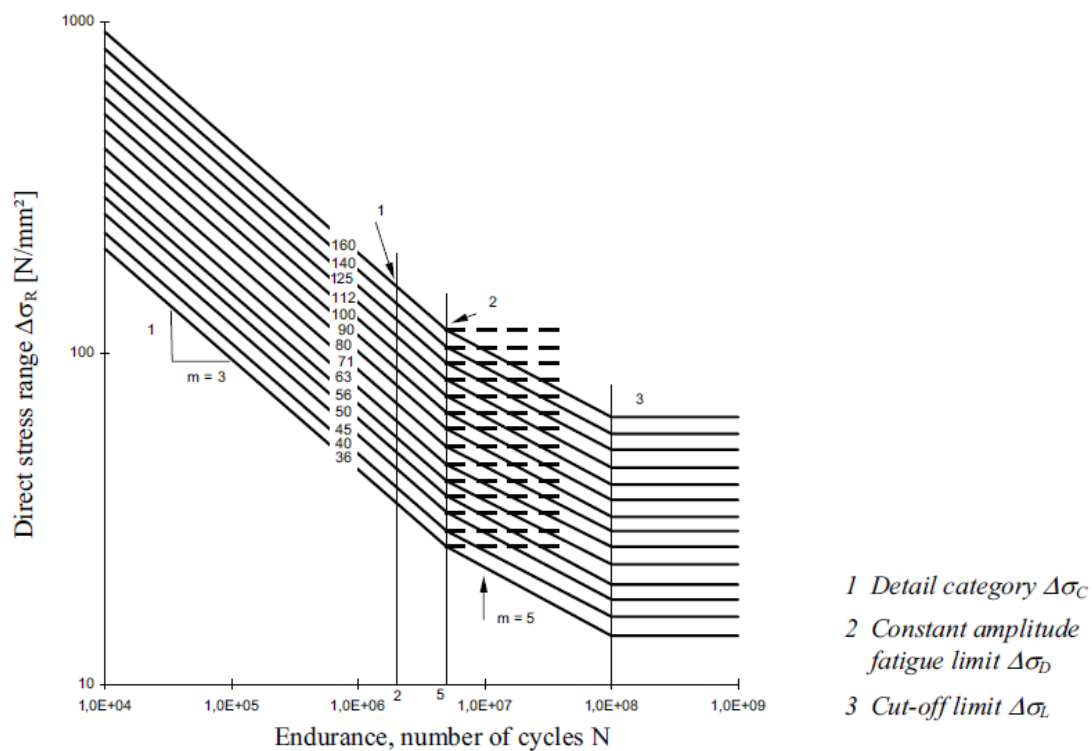


Figure 35 Fatigue strength curves for direct stress ranges [12]

According to Eurocode [12], the fatigue strength for nominal stress range is represented by two series:

$$\log \Delta\sigma_R - \log N \quad S - N \text{ curves}$$

and

$$(5.1)$$

$$\log \Delta\tau_R - \log N \quad S - N \text{ curves}$$

These correspond with typical detail categories. "Each detail category is designated by a number which represents in N/mm^2 , the reference value $\Delta\sigma_C$ and $\Delta\tau_C$ for the fatigue strength at 2 million cycles." [12]

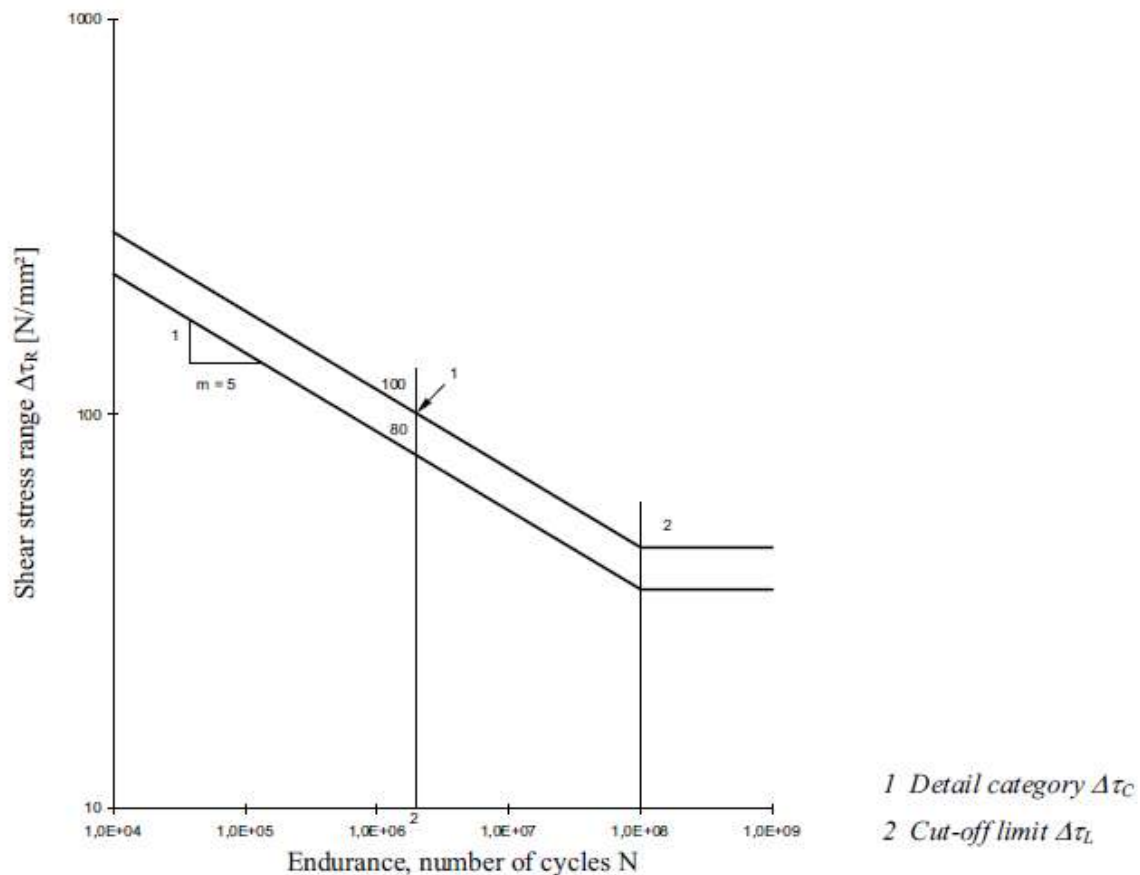


Figure 36 Fatigue strength curve for shear stress ranges [12]

Constant amplitude nominal stresses fatigue strengths:

$$\Delta\sigma^m_R N_R = \Delta\sigma^m_C 5 \cdot 10^6 \text{ with } m = 3 \text{ for } N \leq 10^5 \quad (5.2)$$

$$\Delta\tau^m_R N_R = \Delta\tau^m_C 2 \cdot 10^6 \text{ with } m = 5 \text{ for } N \leq 10^8$$

Constant amplitude fatigue limit:

$$\Delta\sigma_D = \left(\frac{2}{5}\right)^{\frac{1}{3}} \Delta\sigma_C = 0.737 \Delta\sigma_C \quad (5.3)$$

Cut off limit:

$$\Delta\tau_L = \left(\frac{2}{100}\right)^{\frac{1}{5}} \Delta\tau_C = 0.457 \Delta\tau_C \quad (5.4)$$

The statistical analysis of test results on a specific structural element allowed for the definition of one fatigue strength curve [67]. There are many types of structural details, so that all have different strength curves which correspond with the category number.

The fatigue strength curves are based on the experimental investigations for appropriate detail category that considering:

- Stress direction
- Welding and post-welding procedures
- Stress concentration due to the detail geometry
- Crack location
- Local stress concentration due to the size and shape
- Connection types

In the category table [\[12\]](#) classification of structural details are be explained in terms of geometry, model description and requirement.

5.3. Time-dependent failure for steel bridge structural element

A steel plate which is the member of the steel bridge subjected to time-dependent structural failure; fatigue with corrosion. According to the report "Physical research report No. 108", the plate colonized by pitting and surface roughness. The cause can be bolt connection, drop object, water and time.

The time-dependent corrosion must be considered for increasing the degree of accuracy in the fatigue damage assessment. Because corrosion is the consequence that leads the thickness reduction and decreasing the fatigue life, corrosion degradation has in norm effect for reduction of plate thickness due to corrosion wastage.

The study of the Illinois Bridge shows that the fatigue tests after corroded, the fatigue strength of the steel bridge decrease drastically. "Fatigue strength is seriously degraded at high cycle fatigue [\[29\]](#). Normally when the plate is with a good corrosion protection, it can be practical that the fatigue damage is very slight in the initial interval. After the coating life for corrosion models and loading condition is over it can significantly increases the fatigue damage. Time-dependent degradation over the entire service life of the steel bridge structural plate has been verified based on the reduction of plate thickness.

5.4 Factors that affect fatigue-life

Steel bridge structural elements can be fractured at low average tensile stress after a large number of cycles of variable load.

Factors that affect fatigue-life are many, to name some, cyclic stress, geometry, surface quality, material type, environment, residual stresses, temperature and cracking in material.

Environmental conditions can cause erosion, corrosion or gas-phase embrittlement that all affect fatigue life. Corrosion fatigue is a problem encountered in many aggressive environments and also it has effect in normal environment. Certain materials like steel are more level to fatigue, it depends upon the level of humidity and temperature. The life time for steel as notes in the book [27], 10 times greater than aluminum in vacuum. This is mostly due to the effect of the oxygen and water vapor in the air.

5.5 Remaining fatigue life

A damage accumulation calculation method is the most common fatigue life assessment method. The method can handle the situation that has varying amplitude loads. Once the critical structural element is known a calculation of remaining fatigue life can be made. It uses the linear Palmgren-Miner damage rule, which can be simplified:

$$D = \sum \frac{n_i}{N_i} \leq 1 \quad (13.9)$$

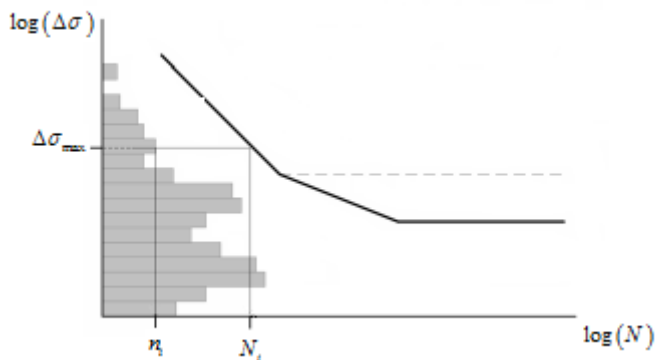


Figure 37 damage accumulation diagram [1]

The construction is assumed to fail when the total damage becomes equal to one or more than one, according to the above equation. To find the damage during service life one should collect the traffic situation with particular time interval, (i.e. with in one year) and one should get the critical structural members that are subjected to structural failure. The analyzed elements/details research result come on the next section.

5.6 Remaining fatigue life of corroded bridge member

A history of stress in a truss member of steel bridge is registered from the passage of traffic over the bridge. The truss members stress history considering load due to passage traffic and time-dependent cross-sectional change due to corrosion.

5.6.1. Corroded bridge member

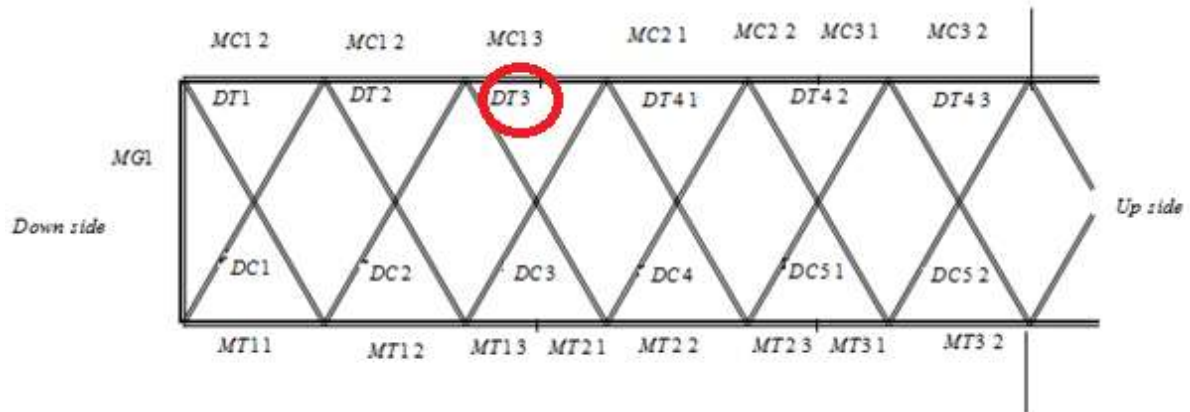


Figure 38 Main truss girder [30]



Figure 39 General view of the riveted railway bridge [30]



Figure 40 Some of corroded location of the bridge [30]

5.6.2. Stress evaluation

The stress time histories can be found through simply multiplying the measured strain data, which means if there are the strain time histories one can analyse the stress time histories. It can be difficult to define a stress cycle directly from the strain time histories graph, sometime it shows many pulses. In this situation one cannot use a cycle counting method. Then, one should use the complex irregular stress time histories into a set of constant stress range frequency data. That is called stress spectrum.

Stress range spectrum and damage accumulation were obtained in the next section. The calculation is based on the net cross-sectional area of the member DT3. [71]

5.6.3. Fatigue life estimation

The evaluation procedure which is recommended [13] for corroded member of steel bridge gives accurate estimates of the remaining mean and safe fatigue life and permits these estimates to be updated in the future to reflect changes in traffic conditions.

The estimation of remaining fatigue life is calculated by using the field measurement data. On the basis of the data on the relevant cross section [71], this section shows tables for the each period that contains, field measurement data, number of stress cycles which are found by using the data and S-N curves for each period.

Relative S-N curve on DNV standards will be:

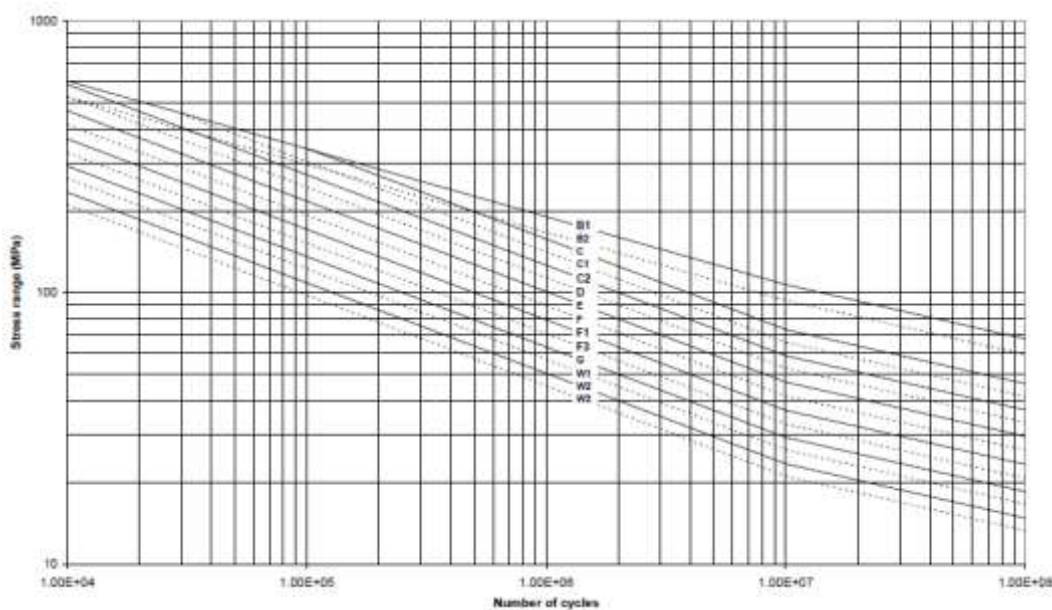


Figure 41 50 S-N curve in air [10]

Fatigue analysis based on S-N data, fatigue-testing specimens in test laboratories derive those data. According the standards, those data have class specification that depending upon:

- the geometrical arrangement of the detail
- the direction of the fluctuating stress relative to the detail
- the method of fabrication and inspection of the detail

a) For the period 1885-1910

$(\Delta\sigma)$ stress range (non corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
35,711	7300	56996176,04	0,000128079
19,057	7300	1276779604	5,71751E-06
$\sum \frac{n_{sr}}{N_{sr}} =$			0,000133796

Table 6 Damage accumulation calculation for the period 1885 – 1910

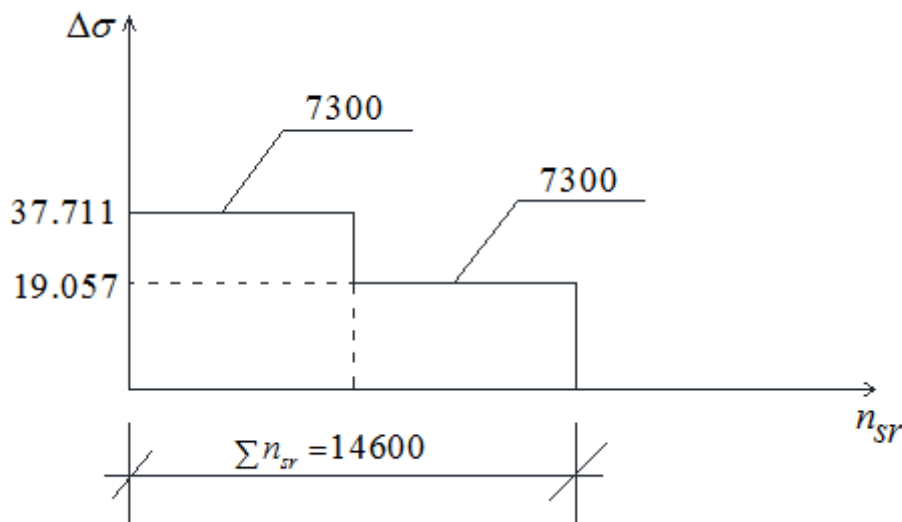


Figure 42 Stress range spectrum for the period 1885 – 1910

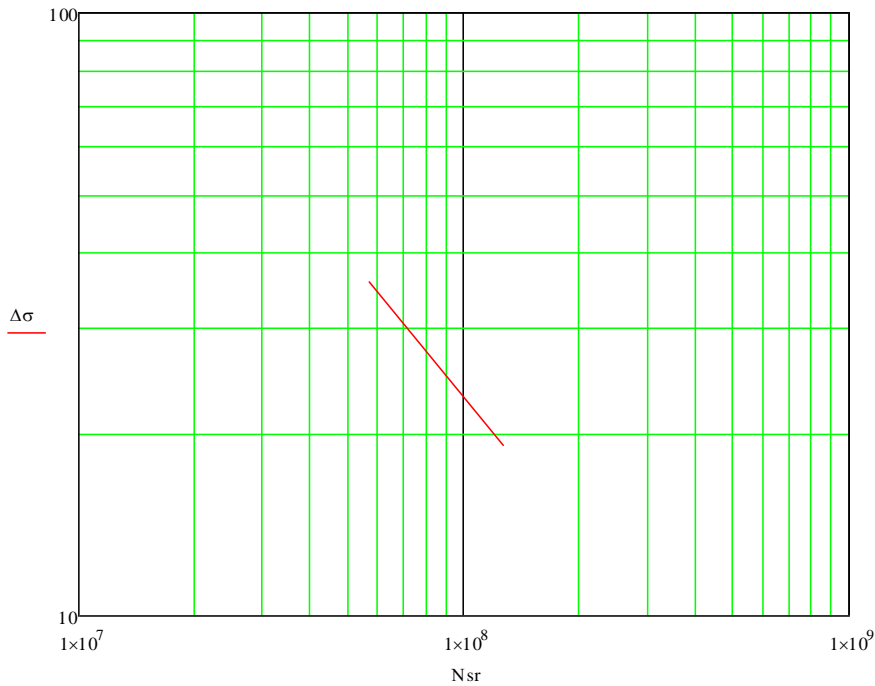


Figure 43 S-N curve for the year 1885-1910 by using Mathcad

b) For the period 1911-1930

$(\Delta\sigma)$ stress range (non corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
35,711	8760	56996176,04	0,000153695
19,057	8760	1276779604	6,86101E-06
$\sum \frac{n_{sr}}{N_{sr}} =$			0,000160556

Table 7 Damage accumulation calculation for the period 1911 – 1930

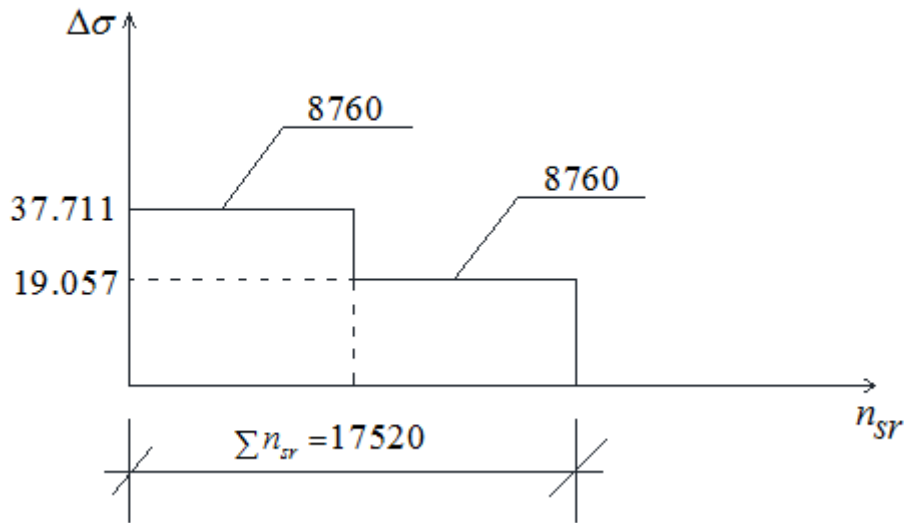


Figure 44 Damage accumulation calculation for each year in the period 1911 -1930

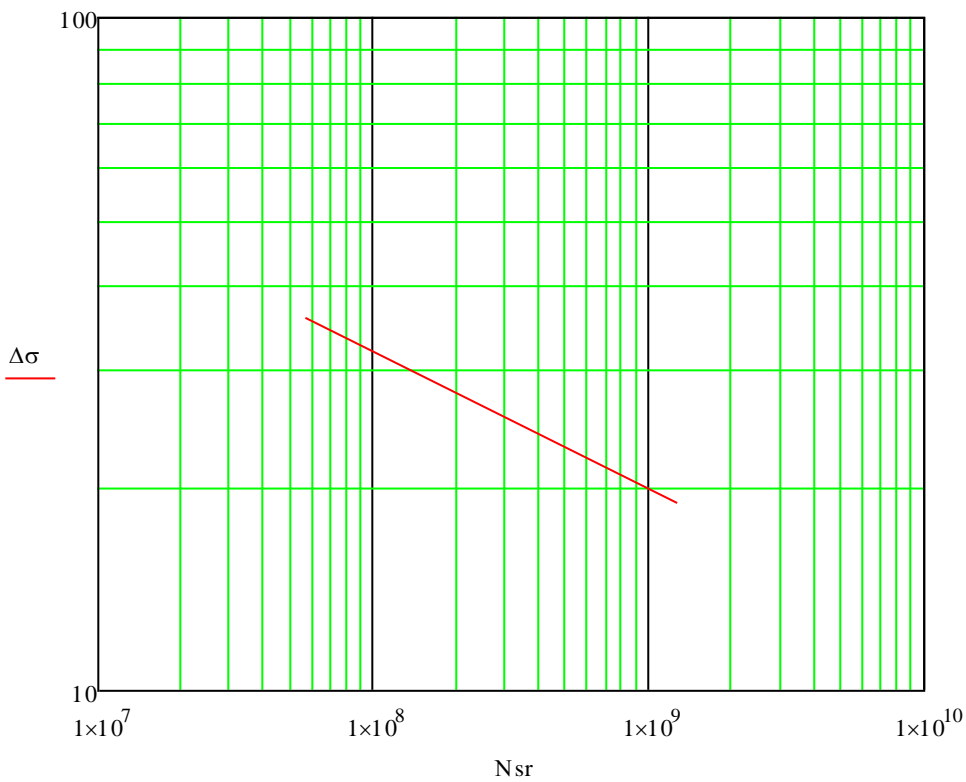


Figure 45 S-N curve between years 1911-1930 by using Mathcad

c) For the period 1931-1950

$(\Delta\sigma)$ stress range (non corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
49,703	8760	11092801,36	0,000789701
35,711	7300	56996176,04	0,000128079
19,057	7300	1276779604	5,71751E-06
$\sum \frac{n_{sr}}{N_{sr}} =$			0,000923498

Table 8 Stress range spectrum for the period 1931-1950

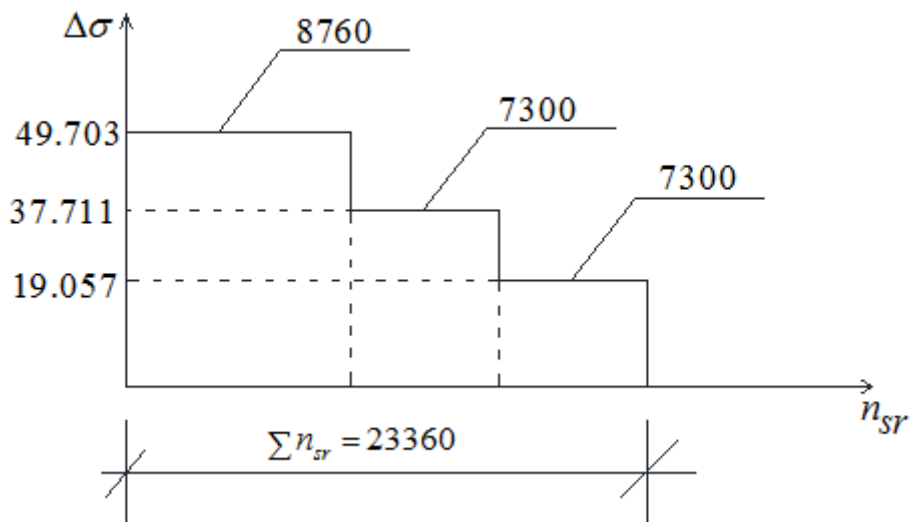


Figure 46 Damage accumulation calculation for each year in the period 1931– 1950

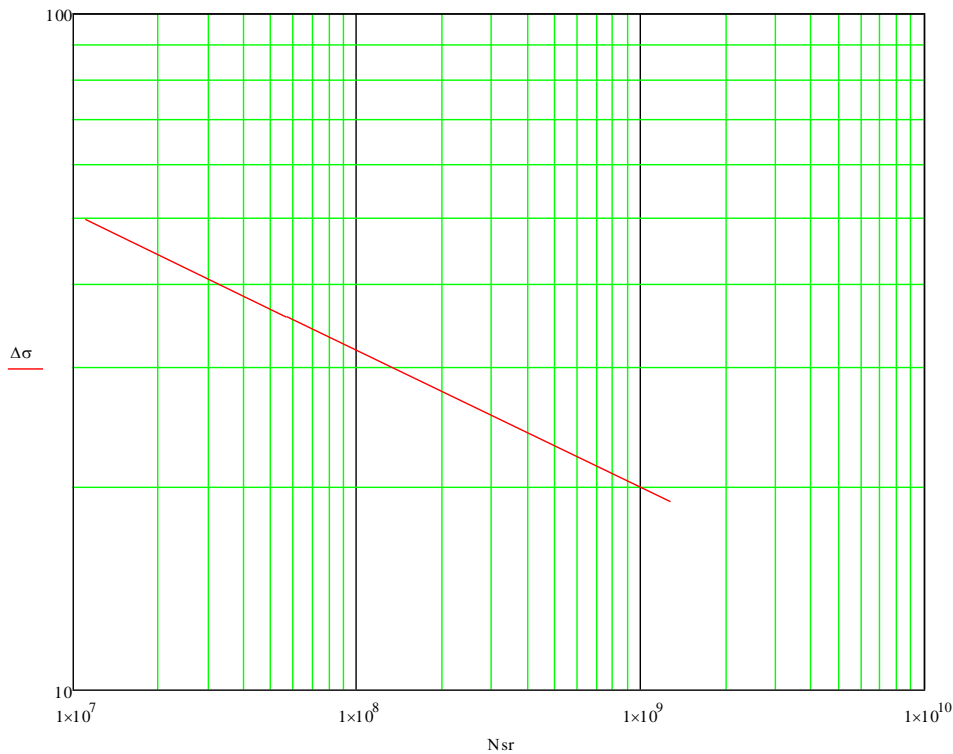


Figure 47 S-N curve between years 1931-1950 by using Mathcad

d) For the period 1951-1970

$(\Delta\sigma)$ stress range (non corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
51,294	5110	9490936,694	0,000538408
33,114	9490	82829472,27	0,000114573
49,703	5840	11092801,36	0,000526468
35,711	5840	56996176,04	0,000102463
19,057	5110	1276779604	4,00226E-06
$\sum \frac{n_{sr}}{N_{sr}} =$			0,001285914

Table 9 Stress range spectrum for the period 1951 – 1970

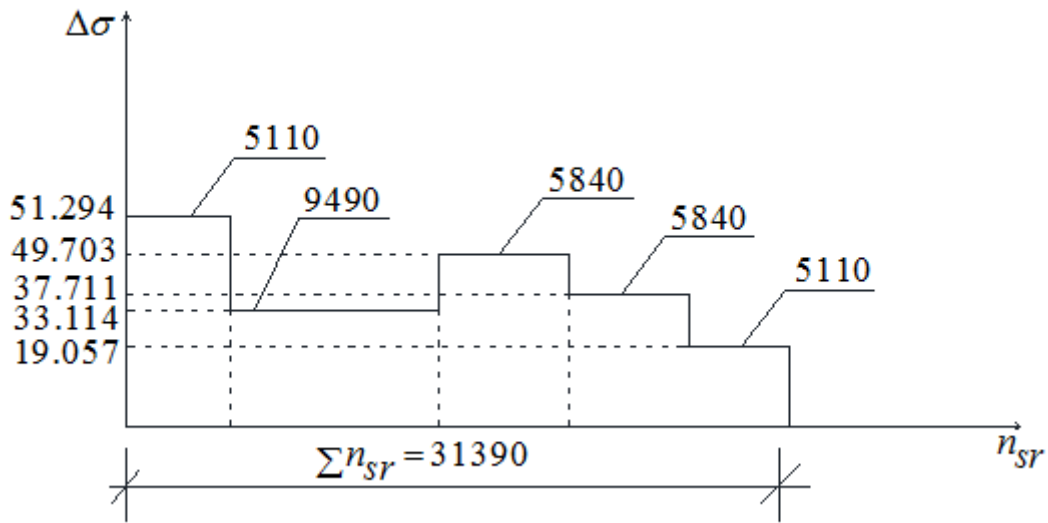


Figure 48 Damage accumulation calculation for each year in the period 1951– 1970



Figure 49 S-N curve between years 1951-1970 by using Mathcad

e) For the period 1971-1975

$(\Delta\sigma)$ stress range (non corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
73,240	3577	1627633,275	0,002197669
63,423	3066	3318788,476	0,000923831
56,488	4599	5887380,964	0,000781162
51,294	1533	9490936,694	0,000161523
42,853	9709	23113161,43	0,000420064
35,711	13797	56996176,04	0,000242069
33,114	4088	82829472,27	4,93544E-05
29,270	511	152567999,7	3,34933E-06
42,853	1022	23113161,43	4,42172E-05
42,853	511	23113161,43	2,21086E-05
32,724	511	87827122,69	5,81825E-06
30,517	2044	124105328,9	1,64699E-05
18,803	30660	1364193813	2,24748E-05
20,258	5110	943403921	5,41656E-06
28,569	2044	172027650,8	1,18818E-05
$\sum \frac{n_{sr}}{N_{sr}} =$			0,004907409

Table 10 Stress range spectrum for the period 1971 – 1975

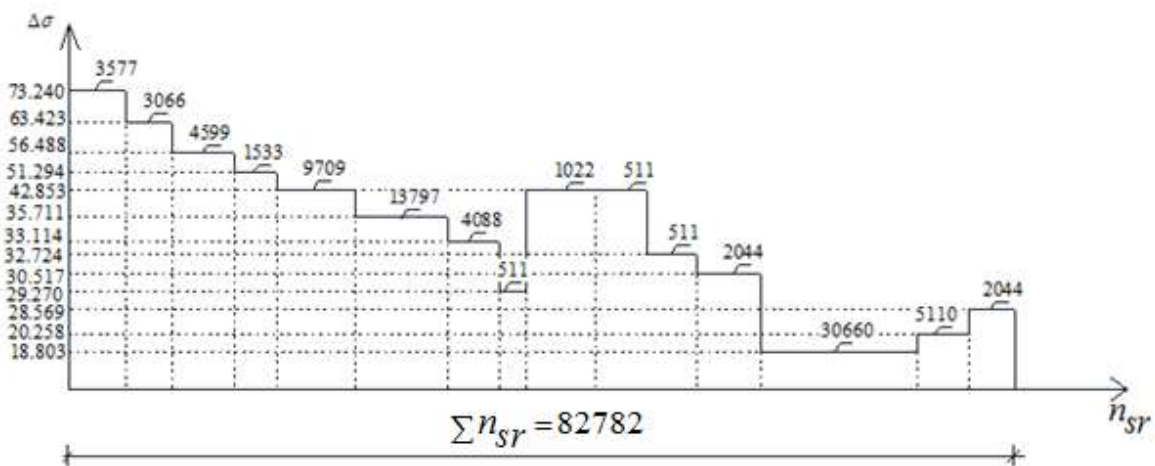


Figure 50 Damage accumulation calculation for each year in the period 1971– 1975



Figure 51 S-N curve between years 1971-1975 by using Mathcad

f) For the period 1976-1985

$(\Delta\sigma)$ stress range (non corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
73,240	4088	1627633,275	0,002511622
63,423	3504	3318788,476	0,001055807
56,488	5256	5887380,964	0,000892757
51,294	1752	9490936,694	0,000184597
42,853	11096	23113161,43	0,000480073
35,711	15768	56996176,04	0,00027665
33,114	4672	82829472,27	5,6405E-05
29,270	584	152567999,7	3,8278E-06
42,853	1168	23113161,43	5,0534E-05
42,853	584	23113161,43	2,5267E-05
32,724	584	87827122,69	6,64943E-06
30,517	2336	124105328,9	1,88227E-05
18,803	35040	1364193813	2,56855E-05
20,258	5840	943403921	6,19035E-06
28,569	2336	172027650,8	1,35792E-05
$\sum \frac{n_{sr}}{N_{sr}} =$			0,005608467

Table 11 Stress range spectrum for the period 1976 – 1985

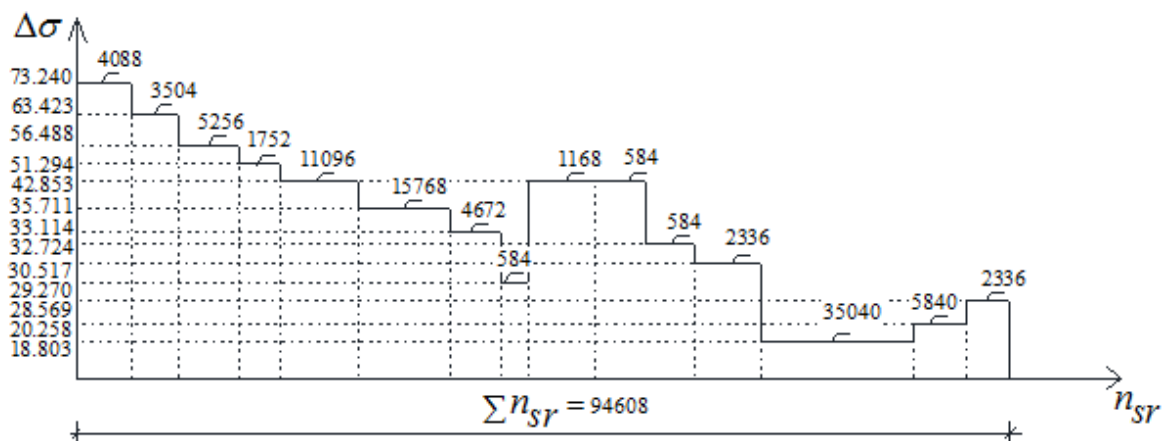


Figure 52 Damage accumulation calculation for each year in the period 1976– 1985

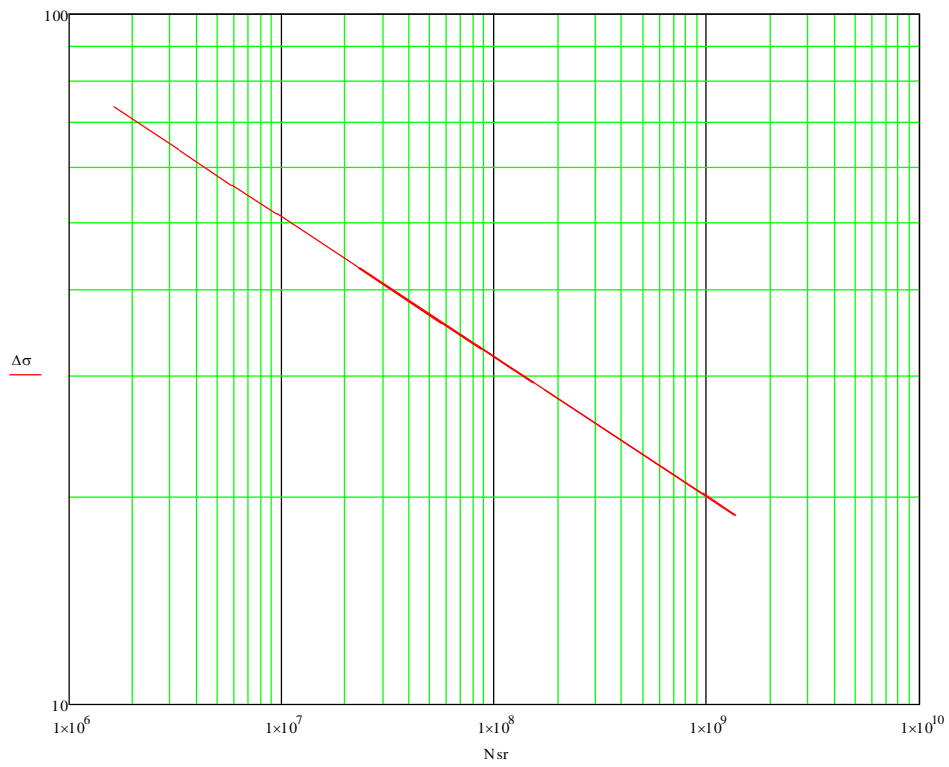


Figure 53 S-N curve between years 1976-1985 by using Mathcad

g) For the period 1986-1995

$(\Delta\sigma)$ stress range (non corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
73,240	4599	1627633,275	0,002825575
63,423	3942	3318788,476	0,001187783
56,488	5913	5887380,964	0,001004352
51,294	1971	9490936,694	0,000207672
42,853	12483	23113161,43	0,000540082
35,711	17739	56996176,04	0,000311231
33,114	5256	82829472,27	6,34557E-05
29,270	657	152567999,7	4,30628E-06
42,853	1314	23113161,43	5,68507E-05
42,853	657	23113161,43	2,84254E-05
32,724	657	87827122,69	7,4806E-06
30,517	2628	124105328,9	2,11756E-05
18,803	39420	1364193813	2,88962E-05
20,258	6570	943403921	6,96414E-06
28,569	2628	172027650,8	1,52766E-05
$\sum \frac{n_{sr}}{N_{sr}} =$			0,006309526

Table 12 Stress range spectrum for the period 1986 – 1995

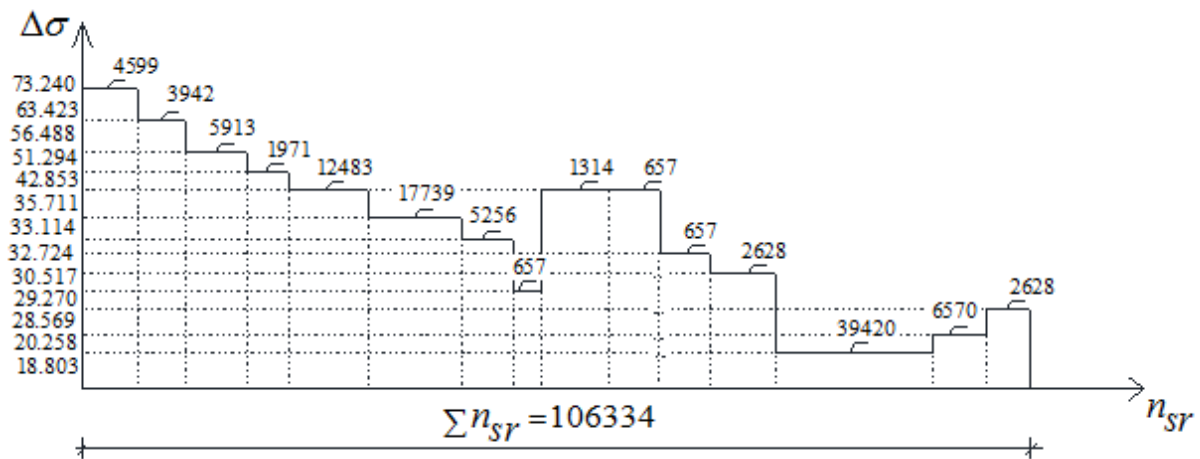


Figure 54 Damage accumulation calculation for each year in the period 1986– 1995

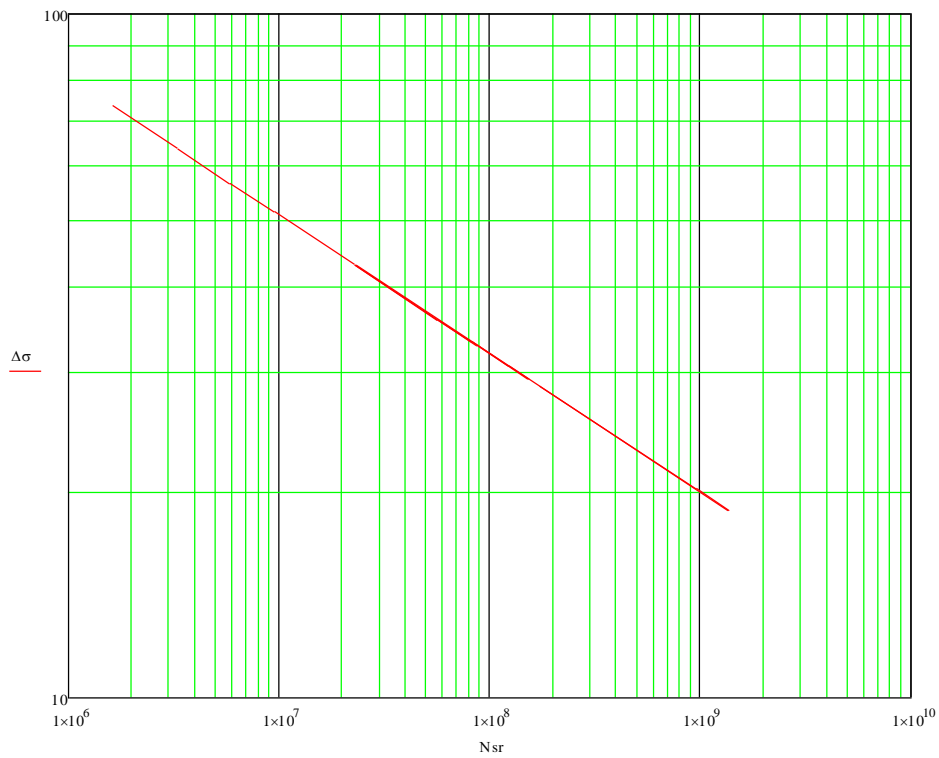


Figure 55 S-N curve between years 1986-1995 by using Mathcad

h) For the period 1996-2001

$(\Delta\sigma)$ stress range (non corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
73,240	5110	1627633,275	0,003139528
63,423	4380	3318788,476	0,001319759
56,488	6570	5887380,964	0,001115946
51,294	2190	9490936,694	0,000230746
42,853	13870	23113161,43	0,000600091
35,711	19710	56996176,04	0,000345813
33,114	5840	82829472,27	7,05063E-05
29,270	730	152567999,7	4,78475E-06
42,853	1460	23113161,43	6,31675E-05
42,853	730	23113161,43	3,15837E-05
32,724	730	87827122,69	8,31178E-06
30,517	2920	124105328,9	2,35284E-05
18,803	43800	1364193813	3,21069E-05
20,258	7300	943403921	7,73794E-06
28,569	2920	172027650,8	1,6974E-05
$\sum \frac{n_{sr}}{N_{sr}} =$			0,007010584

Table 13 Stress range spectrum for the period 1996 – 2001

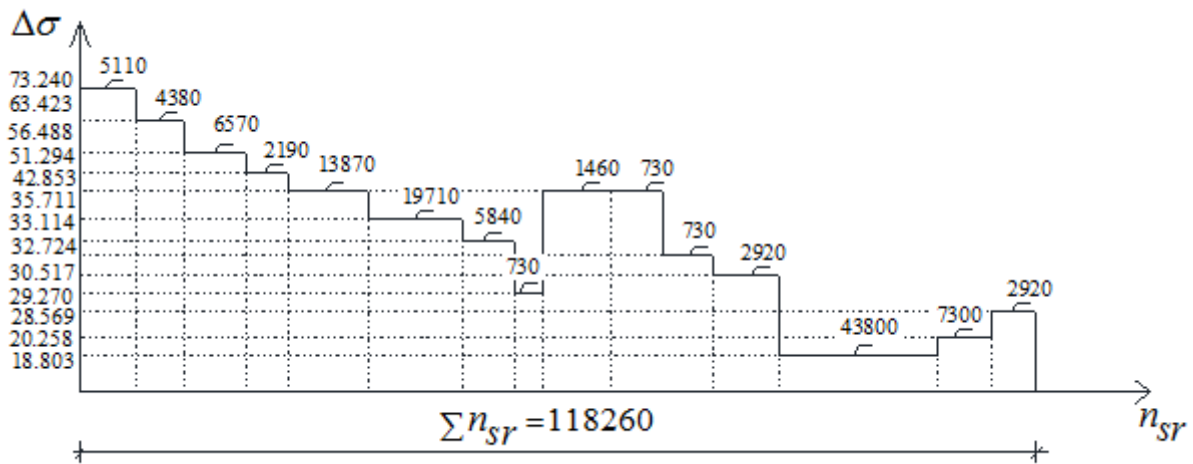


Figure 56 Damage accumulation calculation for each year in the period 1996– 2001

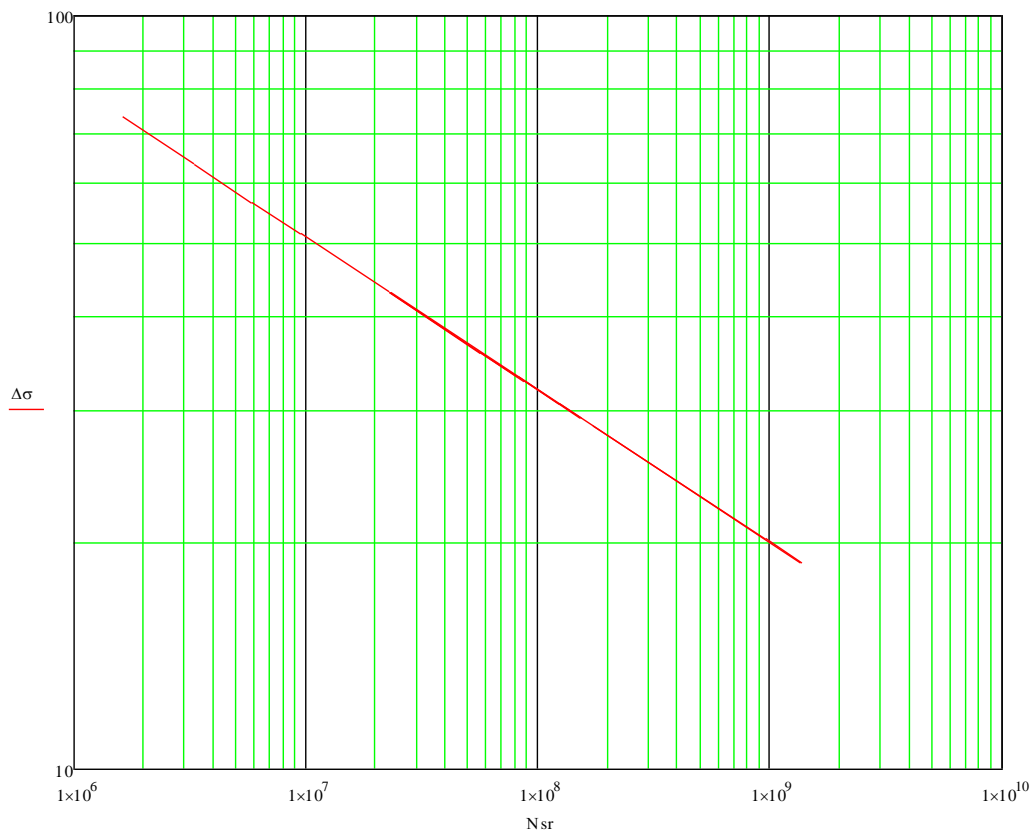


Figure 57 S-N curve between years 1996-2001 by using Mathcad

i) For the period 2001-now

$(\Delta\sigma)$ stress range (non corr)	stress range (corr)	(n_{sr}) cycles/year	(N_{sr}) number of stress cycles	(n_{sr}/N_{sr})
73,240	73,47836969	5110	831198,587	0,006147749
63,423	63,62914142	4380	1694836,51	0,00258432
56,488	56,67214684	6570	3006563,46	0,002185219
51,294	51,46091495	2190	4846824,71	0,000451842
42,853	42,99266312	13870	11803412,6	0,001175084
35,711	35,82721927	19710	29106766,1	0,000677162
33,114	33,22160332	5840	42299295,2	0,000138064
29,270	29,36529172	730	77913316,1	9,36939E-06
42,853	42,99266312	1460	11803412,6	0,000123693
42,853	42,99266312	730	11803412,6	6,18465E-05
32,724	32,83076093	730	44851491,7	1,62759E-05
30,517	30,61598737	2920	63378019,8	4,60728E-05
18,803	18,86465945	43800	696665512	6,28709E-05
20,258	20,32380438	7300	481776834	1,51522E-05
28,569	28,66177541	2920	87850956,7	3,32381E-05

$$\sum \frac{n_{sr}}{N_{sr}} = \mathbf{0,013727959}$$

Table 14 Stress range spectrum for the period 2001- now

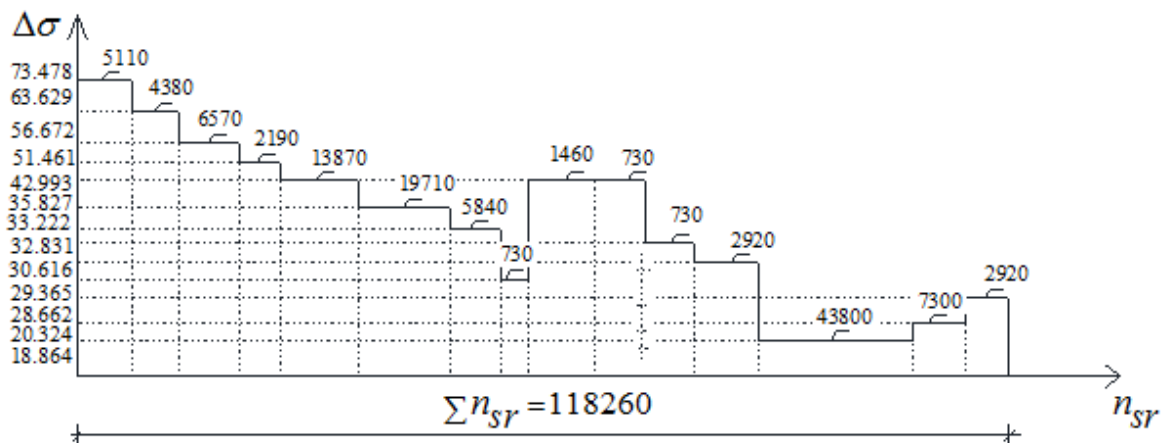


Figure 58 Damage accumulation calculation for each year in the period 2001-now

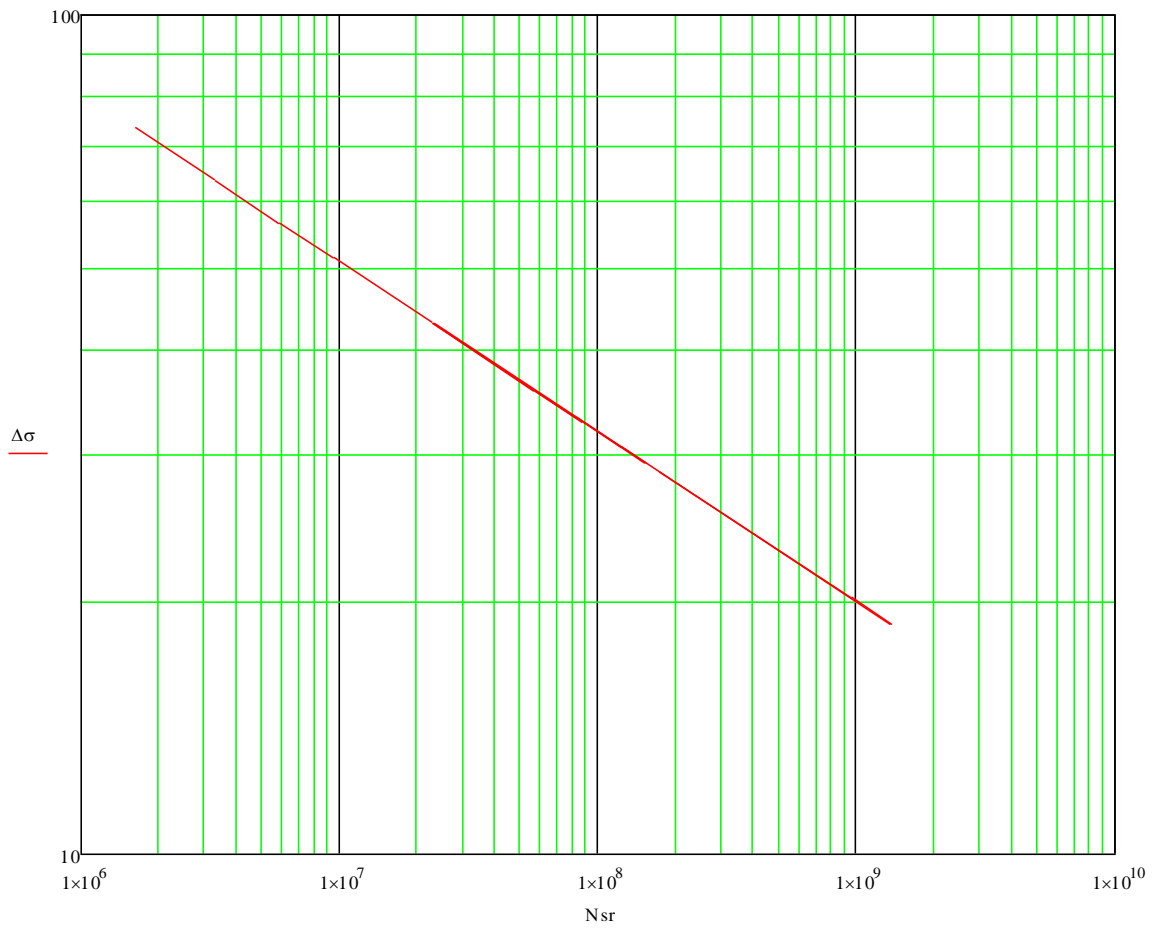


Figure 59 S-N curve between years 1976-1985 by using Mathcad

5.7. Remaining fatigue life

The steel bridges damage during service life find by using all tables that shows damage accumulation. It is done in excel:

Remaining fatigue life (without considering corrosion)	99,9 Years
Remaining fatigue life (considering uniform corrosion only)	79 Years
Remaining fatigue life (Considering uniform +pitting corrosion)	51,02 Years

5.8. Explanation of remaining fatigue analysis in excel worksheet

In this section it will be explained how fatigue analysis in excel worksheet is calculated. It is attempted to give picture of how this worksheet is built up and the different functions. It starts from the worksheets that show stress range spectrum:

$$N_{sr} = C \cdot \Delta\sigma^m$$

Whereas:

C is fatigue strength coefficient

m is fatigue strength exponent for this paper, with the mean slope for redundant and non-redundant classifications taken $m = -4$

To see how remaining fatigue life:

For service life, the years 1885-2001 assumed the steel bridge member is not corroded. Total fatigue damage until 2001 (T_{fd}):

$$T_{fd} = \sum \sum \frac{n_{sr}}{N_{sr}} \cdot \text{the lifetime} = 0.22961998$$

To find remaining damage (R_d):

$$R_d = 1 - T_{fd} = 0.70038002$$

Remaining fatigue life without corrosion (R_{FL}) is $R_{FL} = \frac{R_d}{\frac{n_{sr}}{N_{sr}}} = 99.9032$, $\frac{n_{sr}}{N_{sr}}$ [85]

Remaining fatigue life with uniform corrosion (R_{FLU}) is $R_{FLU} = R_{FL} - (R_{FL} \cdot 0.209) = 79$, 0.209 is thickness reduction per year [2.3.3].

Remaining fatigue life with uniform and pitting corrosion (R_{FLU+P}) is $R_{FLU+P} = \frac{R_d}{\frac{n_{sr}}{N_{sr}}} = 51.02$, $\frac{n_{sr}}{N_{sr}}$

[85]

CHAPTER 6 DISCUSSION, CONCLUSION AND FARTHER STUDIES

6.1. Discussion

The old steel bridge may be dimensions to satisfy permanent, variable and environmental loads in all types of limit states according to that time requirement. The load condition in the bridge is not the same as the old day; there is also a variation on the environment condition. Because of the above reason, assessment of the remaining fatigue life of old steel bridge members cannot be easy. There are many thing can make it be difficult like investigation of origin (i.e. material properties, joint strength) and investigation of operation (i.e. damages, corrosion experience, modifications) to name some.

The information of pitting corrosion is very poor, it is not enough to study construction steel. This is not only in a book but also on standards. Pitting corrosion has not only offshore problem, it has a huge effect for steel construction. There are some, but not with deep explanation. Most of the research works done on rust free steel are in use mostly on offshore structures and most of them focus on how to get a material that has a good pitting corrosion resistance for offshore environment. To some extent it is normal using construction steel both in offshore and onshore, for a short time offshore and most of the onshore structures are constructed by construction steel. Therefore the structural standards and recommended documents ought to have more research about pit.

Verification of aged steel structural critical details are very important in terms of risk and safety. In this paper the critical plates are selected and modeled by the program SAP2000 and the reduction of the plate thickness is received from the report.

Steel bridge structural plates were modeled in the program SAP2000. The stress was analyzed against a code check the program contains. The stress capacity of the plate shows high stress near to the hole. This may be because of the boundary condition and the plate is designed as multiple parts to get the appropriate geometry/shape.

6.2. Conclusion

The purpose of the study was to verify the remaining fatigue life of the steel bridge structural member. In general aged construction is exposed to conditions of stress and environment that ultimately will degrade the material fabric from its initial state. A material strength decreases time to time until the material reaches a state that it is judged to be no longer fit for service. Therefore all type of constructions have to have controlling within time interval.

The result of remaining fatigue life of the steel bridge member in this thesis show the member has remaining fatigue life. It has 79 years remaining fatigue life by considering general corrosion. Whereas it has 51 years remaining fatigue life by considering uniform corrosion plus pitting corrosion. According to this result one can see the steel structure members fatigue life reduce when it subjected to both types of corrosion and it tells the reduction of the material cross-section lead to reduction of fatigue life. Therefor the design of structural members most give more focus on consider those structural failure is necessary.

6.3 Farther studies

It is recommended that more studies shall carried by using laboratory that can be test in terms of pitting corrosion and corrosion fatigue, which consider structural method of analysis. It gives more understanding about steel structure member's failure and it gives the answer why one most consider combination of fatigue and corrosion in structural design. Then it will be easy to give recommendation and suggestion the risk of corrosion fatigue and pitting corrosion for all type of steel structures.

In addition, all standards shall be update by considering those structural failures.

References

1. Dimensjonering av stålkonstruksjoner 2.utg 2010
2. Corrosion for science and engineering second edition
3. Fracture mechanics fundamentals and application 3rd edition 2005
4. Lassen T. Del 1: Elementær styrkeanalyse og dimensjonering, 2003 UIA
5. Lassen T. Del 0: Repetisjon av statikk og fasthets lære, 2007 UIA
6. NS-EN 1993-1-5 2006
7. NS-EN 1993-1-1 2005
8. NS-EN 1993-1-8 2005
9. NS-EN 1993-1-9 2005
10. DNV-RP-C203 2012
11. NS-EN10025-1 2005
12. NS-EN 1991-2 2010
13. Recommendations for estimation of remaining fatigue life
14. Materials science and engineering eighth edition
15. Fatigue of materials second edition
16. Corrosion understanding the basics (2000)
17. Fatigue damage, crack growth and life prediction (1997)
18. Design of fatigue and fracture resistant structures (1982)
19. Stål håndbok del 3 2 utgave (1994)
20. <http://www.materialsengineer.com/CA-Wear.htm>
21. <http://corrosioncontrol.wikispaces.com/>
22. <http://corrosion-doctors.org/Forms-pitting/pit-depth.htm>
23. Almar - Næss, A. (1985). Fatigue Handbook - Offshore Steel Structure
24. Advanced Mechanics of materials
25. Norsok standard N-004
26. NS-EN 1993-6 2005
27. Steel: microstructure and properties
28. http://fcp.mechse.illinois.edu/media/pdfs/1_Notches.pdf
29. Physical research report No. 108
30. Remaining fatigue life estimation of corroded bridge members
31. Uhlig's Corrosion handbook
32. Staten vegvesen 2009

APPENDIX

A.1. SAP 2000

A.1.1 General

Sap 2000 is structural engineering modulating and analysis software. It can use to model, analyze, design and display the structural geometry, properties and analysis results. In general the program uses finite element analysis method which performs the static or dynamic, linear or nonlinear dynamic analysis. To construct the geometry that is not easy in SAP2000, the program contributes with AutoCAD are used. That can done by modelling the frame on AutoCAD and converting it to SAP2000. That can help to model different geometry, which cannot find on SAP 2000 library. For example, plates with holes are not easy to model on SAP 2000, but by modelling on AutoCAD be easily modeled and analyses on SAP 2000.

A.1.2. Dimension of the chosen plate

In this section one can see the dimension of the plate and the name of specimen that is using to show how the program works. [\[1.3\]](#).

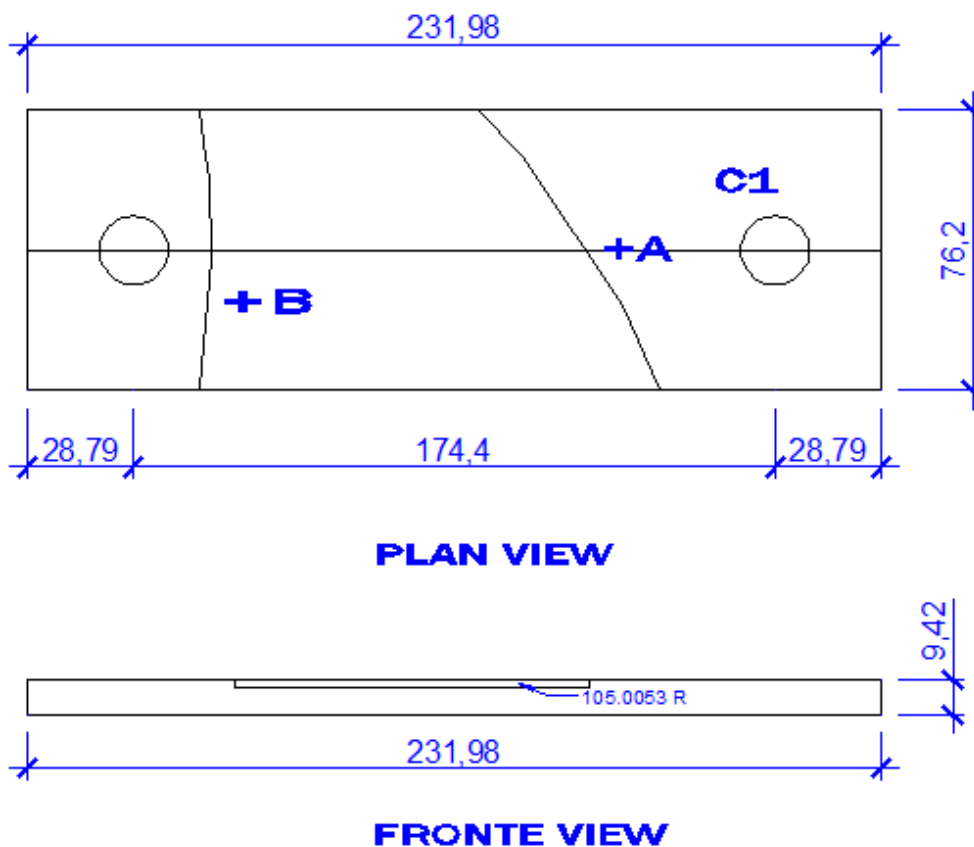


Figure 1 Dimension and pitting in specimen C1

A.1.3. Example of SAP 2000 model and analyses

The simulation in this section will perform in order to show how the program works.

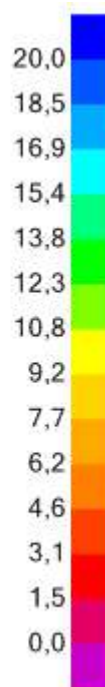


Figure 2 Actual stress simulation



Figure 3 3D model

BEGIN ANALYSIS

2014/06/18 17:33:37

RUNNING ANALYSIS WITHIN THE GUI PROCESS
USING THE ADVANCED SOLVER (PROVIDES LIMITED INSTABILITY INFORMATION)

NUMBER OF JOINTS	=	228
WITH RESTRAINTS	=	99
NUMBER OF SOLID ELEMENTS	=	76
NUMBER OF LOAD PATTERNS	=	2
NUMBER OF ACCELERATION LOADS	=	6
NUMBER OF LOAD CASES	=	1

ELEMENT FORMATION

17:33:37

LINEAR EQUATION SOLUTION

17:33:37

FORMING STIFFNESS AT ZERO (UNSTRESSED) INITIAL CONDITIONS

TOTAL NUMBER OF EQUILIBRIUM EQUATIONS	=	533
NUMBER OF NON-ZERO STIFFNESS TERMS	=	9850
NUMBER OF EIGENVALUES BELOW SHIFT	=	0

LINEAR STATIC CASES

17:33:37

USING STIFFNESS AT ZERO (UNSTRESSED) INITIAL CONDITIONS

TOTAL NUMBER OF CASES TO SOLVE	=	1
NUMBER OF CASES TO SOLVE PER BLOCK	=	1

LINEAR STATIC CASES TO BE SOLVED:

CASE: TENSILE

.....
ANALYSIS COMPLETE

..... 2014/06/18 17:33:37

Figure 4 Analysis

A.2. Plate theory

A plate is a structural element with small thickness that is relative to the length. Usually the load-applied normal to the plate will give bending in the middle of the plate. If one looks closely at a dice taken out of the plate one can see that exchange between the tensile and compressive stresses with maximum values in the upper and lower edge of the plate.

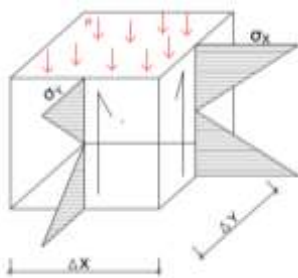


Figure 5 Stresses on a dice that taken [4]

Although the loads are constant the deflection and stresses in a large plate flat become so large that it will have to put stiffeners to the plate.

Below is shown a picture of a stiffening plate flat. This flat plate comprised principally:

- (1) Load bearing
- (2) Stiffer
- (3) Flat plate

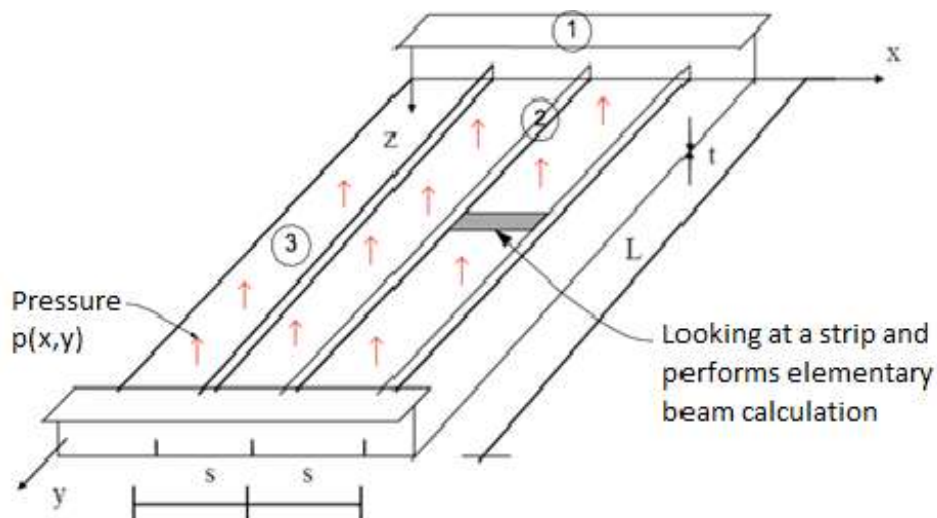


Figure 6 Illustration of a flat plate with stiffer and load bearing [4]

The plates that are stored at stiffeners, which are stored on the supports, take up the pressure p . That pressure press downs the plate. The plate hangs on supports and supports leads forces through stiffeners. The x axis of the plate can be taken in any one reference direction and the y axis is taken in the direction normal to the x direction. The plate length and breadth are denoted by L and S , respectively. The plate thickness is t .

The material of plates in steel plated structures is normally has yield strength, σ_y , being typically in the range of 230–450 MPa.

Since the support members or stiffeners are located along the plate edges, it is considered that the plate is simply supported, which are kept straight until the ultimate strength is reached.

A simplified method for controlling the bending stress in the stiffening panels is called plate strip method. These involve looking for a little “strip” of the disc and perform an elementary beam calculation.

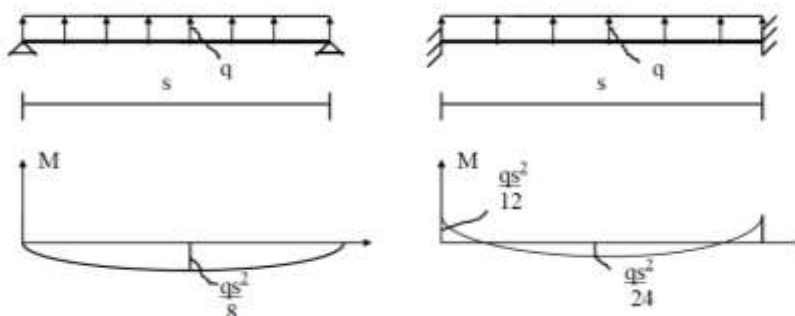


Figure 7 Moment diagram for a plate strip [4]

Each strip acts as a small beam with support at each end and the cross-sectional width of one; height t and pressure p is considered a regular distributed load on the beam. Supports to this beam will in fact be neither freely supported nor fixed support but something between these two. A will in principle always be on the safe side, select the beam as simply supported. If the plate area has a right next flat across the stiffener with the same strain it could be appropriate to calculate the rigid clamping. If there is no next felt or if next felt is not charged, expected beam can be simply a supported beam.

Bending stress dominates here:-

$$\sigma_x = \frac{M}{I} \cdot \frac{t}{2} = \frac{\frac{\rho \cdot d \cdot s^2}{8}}{\frac{d \cdot t^3}{12}} \cdot \frac{t}{2} = \frac{3 \cdot \rho \cdot s^2}{16t^2} \quad (\text{A.2.1})$$

It is assumed therefore that: $\sigma_x < \frac{f_y}{\gamma_M}$

According to P. Boresi [24], the plate can be classified in terms of the magnitude of the thickness compared to the magnitude of the other dimensions. To see the deflections of a plate, the deflection of relatively thick plate have very small deflection, relatively thin plates have small deflection and very thin plates have large deflection. Stresses of a dice taken of a plate The theory of that has a small deflection with a limit to cases in which the lateral displacement ω of the plate in z direction is less than about half of the plate thickness h.

Timoshenko and Woinowsky-Keieger in 1959 have presented a large collection of solved rectangular plate problems. When bending effects are dominant for rectangular plate, the solution of a plate problem:

$$\nabla^2 \nabla^2 \omega = \frac{p}{D}$$

$p \rightarrow$ lateral pressure

$D \rightarrow$ flexural rigidity

The reduced stress method can be used to determine the stress limits for stiffened or unstiffened plates. The method is useful for all type cross sections, not only for standard steel plate cross section. The cross section has full effect until it reach the stress limit. This method uses von Mises Criterion to verify the interaction between different stress type. An interaction equation is essential, the verification of each load type followed by a combination of these load type.

The verification of the plate subject to the complete stress field

$$\sqrt{\left(\frac{\sigma_{xED}}{p_x \cdot \frac{f_y}{\gamma_{M1}}}\right)^2 + \left(\frac{\sigma_{zED}}{p_z \cdot \frac{f_y}{\gamma_{M1}}}\right)^2 - \left(\frac{\sigma_{xED}}{p_x \cdot \frac{f_y}{\gamma_{M1}}}\right) \cdot \left(\frac{\sigma_{zED}}{p_z \cdot \frac{f_y}{\gamma_{M1}}}\right) + \left(\frac{\tau_{ED}}{\chi_M \cdot \frac{f_y}{\gamma_{M1}}}\right)^2}$$

A.2.1 Dimensioning of plates

In general dimensioning of steel plates is strategically used standards, which has particular requirement. The standard can tell how to manage all types of capacity. For example, plate with an axially pressure can be dimensioned by using differential equation:

$$\nabla^4 w = \frac{P}{D} \quad (A.2.2)$$

$$w(x, y) = X_n(x) \sin \frac{n\pi y}{b}$$

n is arbitrary number for boundary conditions at y=0 and y=b

$$\left. \begin{aligned} w &= 0 \\ M_{yy} &= D(w_{yy} + \nu w_{xx}) = 0 \end{aligned} \right\} \text{at } y = 0, b$$

$$w(x, y) = Y_n(y) \sin \frac{n\pi x}{a}$$

$$\left. \begin{aligned} w &= 0 \\ M_{xx} &= D(w_{xx} + \nu w_{yy}) = 0 \end{aligned} \right\} \text{at } x = 0, a$$

This can simplified:

$$\nabla^4 w + \frac{N_x \partial^2 w}{D \partial x^2} = 0 \quad (A.2.3)$$

For a rectangular plate that is simply supported plate with an axial pressure and boundary conditions finding critical stress can done: [\[6\]](#)

$$\sigma_{cr} = k_{\sigma} \cdot \sigma_E \quad (A.2.4)$$

$$\sigma_E = \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b} \right)^2$$

$$\therefore \sigma_{cr} = k_{\sigma} \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b} \right)^2$$

σ_E is depends on the material E-model, poses ratio and plats geometry.

k_{σ} is depends on the load distribution and boundary conditions, and determined from diagram

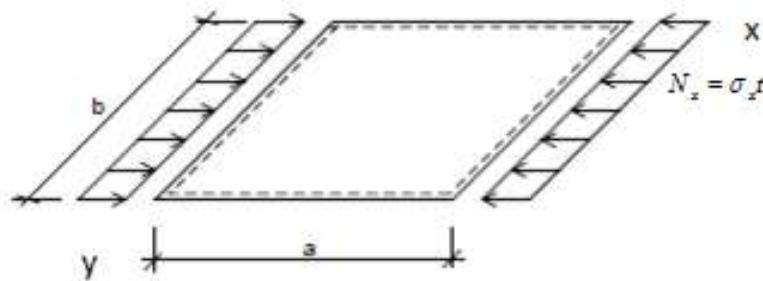


Figure 8 A critical stresses at a plate [1]

As shown on the figure [Error! Bookmark not defined.] plate has a useful critical capacity.

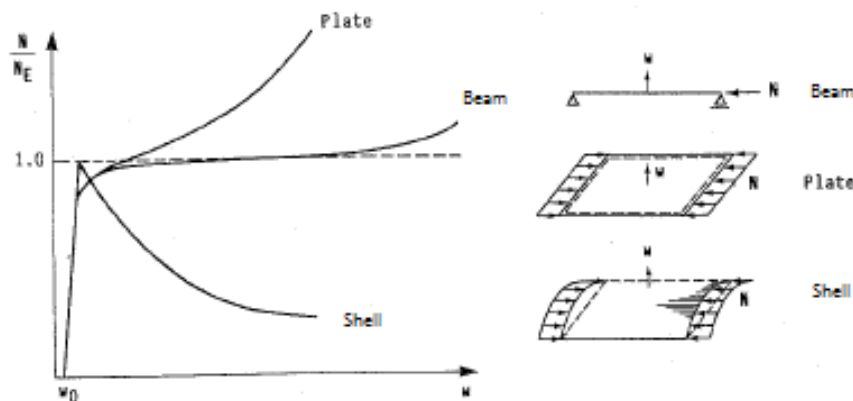


Figure 9 Characteristic behavior of the pressure components [6]

Steel bridge structure members (plates) that are in critical mode of fatigue failure shall be investigated with respect to fatigue. The design of steel plate can be done using two type of

design methods according to Eurocode 3 [6]. Those are effective width method and reduced stress method. Comprising resistance models for shear force and transverse force in effective width method is very efficient for standard geometries. The reason for this is the method accounts not only for the post-critical reserve in a single plate element but also for load shedding between cross sectional elements. On the other hand, reduced stress method abstains from load shedding between cross sectional elements, but it fully accounts for the post-critical reserve in a single plate.

A.2.2. Von Karman plate theory

The theory of von Karman has used the nonlinear equation for elastic plates, it was proposed in 1910 by von Karman. Assumption of the von Karman theory, the plate is thin to complete the length of the plate. And also the edge region, each of the width $\frac{b_e}{2}$, carries the stress up to the yield while the center is stress free.

Replacing the real plate of width (b) (with varying stress N_x (y)) with a model plate with a width (b_e) and constant stress, so that the plates have the same capacity.

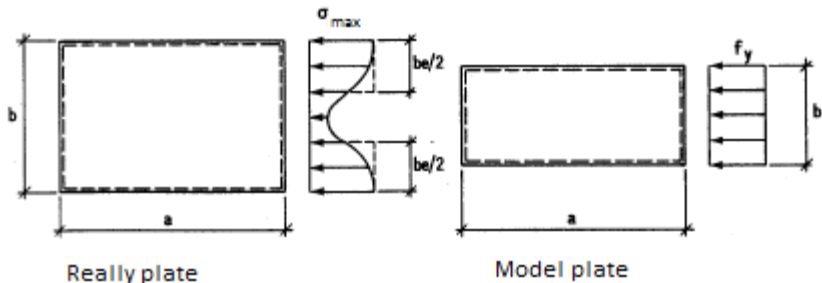


Figure 60 Illustration of plate replaces [6]

Fracture Criterion

Model plate capacity is fully utilized when its critical stress $f_{x,cr}(b_e)$ is equal to the yield stress f_y .

$$f_{x,cr}(b_e) = k_\sigma \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b_e}\right)^2 = f_y \quad (A.2.5)$$

or

$$f_{x,cr}(b_e) = k_\sigma \frac{\pi^2 E}{12(1-\nu^2)} \left(\frac{t}{b}\right)^2 \left(\frac{t}{b_e}\right)^2 = f_{x,cr}(b) \cdot \left(\frac{b}{b_e}\right)^2 = f_y$$

$f_{x,cr}(b_e)$ is Euler's stress of the real plate. For steel:

$$f_{x,cr}(b_e) = k_\sigma \frac{\pi^2 2.1 \cdot 10^5}{12(1-0.3^2)} \left(\frac{t}{b}\right)^2 \frac{235 f_y}{235 f_y} = k_\sigma 807.6 \cdot \varepsilon^2 f_y \left(\frac{t}{b}\right)^2 \quad (\text{A.2.6})$$