KAUNAS UNIVERSITY OF TECHNOLOGY
CIVIL ENGINEERING AND ARCHITECTURE FACULTY

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CORROSION EFFECT FOR STEEL BRIDGES STIFFNESS, STABILITY AND SAFE FACTOR

Master's Degree Final Project

Supervisor
Assoc. prof. Dr. Saulius Zadlauskas

KAUNAS, 2018

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Structural and building products engineering (code M6026O21)

"Evaluating corrosion effects in existing steel truss bridge"

DECLARATION OF ACADEMIC INTEGRITY

15 January 20 18
Kaunas

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SUMMARY

The paper analyzes the strength and deformability of the Bridge across the Minija River, built in Lithuania, in assessing the influence of steel corrosion. The bridge over the Minija River was built in 1921 according to German design standards DIN 1072. The bridge is two spans; the bridge overlay consists of two arched type metal trusses.

In this work, theoretical calculations are made of the changes in the stresses and the deflection of the bridge if the cross-sectional area of the metal retaining elements is reduced. After the visual inspection of the bridge, it was found that from 10 to 15 percent there is a decrease in the cross-sectional area of the bearing elements due to corrosion in them. In the work, the safety of the bridge (evaluating the influence of corrosion and other possible factors) was proposed to estimate the safety factor of the bridge. The bridge's safety factor depends on the strength of the bridge, the impact of heavy vehicles, the dynamics of the bridge, the variables and the constant load factors. The thesis provides an in-depth analysis of how the bridge's safety factor varies in different scenarios: if an inspection changes, dynamic loads increase if the overlay dynamic coefficient increases, or if several parameters deteriorate at the same time. At the end of the paper, conclusions and recommendations regarding the strength, deformability and safe operation of the bridge are presented.

SANTRAUKA

Darbe analizuojama Lietuvoje pastatyto tilto per Minijos upę stiprumas ir deformatyvumas įvertinant plieno korozijos įtaką. Tiltas per Minijos upę buvo pastatytas 1921 m. pagal vokiškas tiltų projektavimo normas DIN 1072. Tiltas yra dviejų tarpatramių, tilto perdangą sudaro dvi arkinio tipo metalinės santvaros.

Darbe atlikti teoriniai skaičiavimai, kaip keičiasi įtempiai ir tilto įlinkis, jei sumažinamas metalinių laikančiųjų elementų skerspjūvio plotas. Atlikus tilto vizualią apžiūrą buvo nustatyta, kad nuo 10 iki 15 procentų yra sumažėjęs laikančiųjų elementų skerspjūvio plotas dėl korozijos juose. Darbe pasiūlyta tilto saugą (įvertinat korozijos įtaką ir kitus galimus veiksnius) įvertinti apskaičiuojant tilto saugos faktorių. Tilto saugos faktorius priklauso nuo tilto būklės, sunkiasvorių transporto priemonių sukeliamo poveikio, tilto dinamikos, kintamų ir nuolatinių apkrovų patikimumo koeficientų. Darbe pateikta išsami analizė, kaip keičiasi tilto saugos faktorius esant skirtingiems scenarijams: jei keičiasi apžiūra, jei didėja dinaminės apkrovos, jei didėja perdangos dinamiškumo koeficientas, ar jei prastėja keli parametrai vienu metu. Darbo gale pateiktos išvados ir rekomendacijos dėl tilto stiprumo, deformatyvumo ir saugaus jo eksploatavimo.

1 INTRODUCTION

1.1 BRIDGE FAILURE

Catastrophic failure, with loss of life, is probably the most publicized aspect of bridge corrosion. Collapses of the Point Pleasant (Silver) Bridge over the Ohio River in 1967 and the Mianus River Bridge on Interstate 95 in Connecticut are two widely known bridge failures. The Point Pleasant Bridge, an eye bar chain suspended structure, failed because of corrosion cracks at the pin hole in an eye bar.

Fig 1. The point pleasure silver bridge over the Ohio River in 1967

Fig. 2 Mianus River Bridge on Interstate 95 in Connecticut

a. Corrosion is the major cause of deterioration of steel bridges.

b. The results of this deterioration can range from progressive weakening of a bridge structure over a long period of time to sudden bridge collapse.

c. The effects of corrosion damage vary with the type of structure and the location and extent of deterioration.

d. Corrosion damage must be carefully appraised and evaluated. In some cases, immediate repair or closure is necessary, while in other cases, the conditions created by corrosion can be tolerated. In all cases, however, the likely progression of corrosion must be considered.

AIM

Stiffness, stability and safety factor of the bridge involving corrosion process.

Fig. 3 The elevation of the bridge

2 LITERATURE REVIEW

2.1 CORROSION OF STEEL

"Corrosion of steel is the deterioration and eventual destruction of the metal because of its reaction with the environment. Chemically, it is the transformation of a metal to its oxide through a reaction involving oxygen, water, or other agents. Figure 4 depicts the steel life cycle." [2]

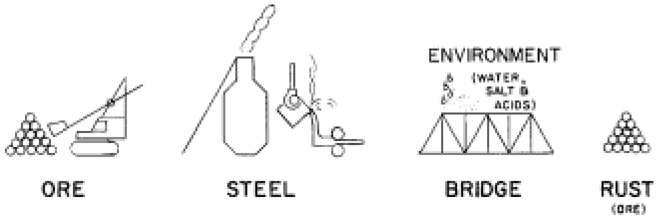


Fig. 4 Steel life cycle [2]

2.2 CORROSION FORMS ON BRIDGES

"Corrosion is known to appear in many forms. These forms are classified according to how the corrosion attacks the metal' the corrosion spectrum ranges from uniform corrosion, which can be identified visually, to stress corrosion, which cannot be identified with the naked eye." [2]

2.3 EIGHT FORMS OF CORROSION HAVE BEEN IDENTIFIED IN FONTANA'S CORROSION ENGINEERING.

2.3.1 UNIFORM CORROSION.

DEFINITION

"Uniform corrosion or rusting (also known as general corrosion) is a general thinning of metalwork in a universal or overall manner. It is a natural process exhibited by all bare metals exposed to the atmosphere. On steel bridges, it is observed as a uniform rust over the entire surface. Uniform corrosion can be identified by the naked eye." [1]

OCCURRENCE

"One of the simplest examples of uniform corrosion is the formation of the oxide product that protects weathering steel. New weathering steel generally is coated with mill scale that eventually flakes off as a result of weathering and corrosion, 8 exposing the base metal. A progression of corrosion occurs until the surface is covered by its own corrosion product. The corrosion product reduces the corrosion rate by forming a barrier between the metal and the environment. Often bridges in arid areas exhibit uniform corrosion because of the lack of moisture which would have caused other forms of corrosion to occur. Uniform corrosion of steel typically consists of many small pits joined together. With the thinning of a paint system, the peaks of metal are exposed and a uniform coating of rust or corrosion occurs. Uniform corrosion is most commonly found on steel

bridges on plates and shapes with large surface areas that can be uniformly attacked or oxidized. Usually these members can dry quickly, preventing other forms of corrosive attack. Such members include girder webs, vertical gusset plates, and truss verticals and diagonals." [1]

2.3.2 GALVANIC CORROSION

DEFINITION

"Galvanic corrosion or dissimilar metal corrosion is caused when metals of different composition are placed together in the presence of an electrolyte. The difference in their corrosive potential produces electron flow, with one of the metals as the anode and one as the cathode. The intensity of corrosion depends not only on the difference in corrosion potential between the metals but also on the ratio of the exposed area of the metals and their specific corrosion behavior. Galvanic corrosion can usually be identified visually." [1]

OCCURRENCE

"Galvanic corrosion most commonly occurs on steel bridges where aluminum light poles, handrails, or electrical conduits are in contact with steel or where galvanized steel is in contact with bare steel (such as weathering steel). Insulating materials are often placed between the metals to prevent the formation of galvanic corrosion. Galvanic corrosion may also occur on steel where mill-scale is exposed. Galvanic corrosion has a beneficial effect in the application of zinc paints on steel. The intent is that the less resistant metal, zinc, will be sacrificed in the corrosion process and the steel surface will remain free of corrosion." [1]

2.3.3 CREVICE CORROSION

DEFINITION

"Crevice corrosion is a form of localized corrosion occurring at confined locations where easy access to the outside environment is prevented. It is caused by differences in the environment inside and outside of the crevice, such as concentrations of oxygen cells or metal ion cells. The presence of chloride ions also promotes crevice corrosion. Crevice corrosion can usually be visually observed." [1]

OCCURRENCE

"Crevice corrosion is one of the most common forms of corrosion found on steel bridges. It occurs within gaps between mating surfaces as small as several thousandths of an inch wide, such as along edge openings of built-up members with multiple plies of plates, between back-to-back angles used for bracing members, between lacing bars and adjoining components, and between closely spaced eye bars. Crevice corrosion can also occur between steel and other materials, such as timber decks or concrete slabs. These gaps are commonly formed by variations in thickness or alignment from mill rolling of plates and shapes, shearing of plate edges in the fabrication process, and excessive spacing of fasteners that fail to seal the components with their clamping action. Steels that rely on an oxide film for protection, such as weathering steel, are particularly susceptible to crevice corrosion. These films are destroyed by the high concentrations of chloride or hydrogen ions that can occur in crevices." [1]

2.3.4 DEPOSIT ATTACK

DEFINITION

“Deposit attack is a localized corrosion of the crevice corrosion form caused by a deposit of foreign material acting as a shield to create a confined space that behaves like a crevice. These deposits can also hold moisture, which provides an electrolyte. Deposit attack can be observed visually.” [1]

OCCURRENCE

“Deposit attack frequently occurs on bridges at locations of debris deposits harboring moisture. The debris often consists of road dirt or trash deposited on horizontal surfaces either by wind or by water draining off the roadway. The debris deposits can have a local source, such as coal dust in mining areas, grain or other by-products in farm regions, or salts from deicing agents in northern or high altitude regions. Pack rust itself can act as a deposit and promote further corrosion. One of the most annoying types of deposits comes from bird nests and bird excrement. Many of the materials deposited contain very active agents that accelerate corrosion. Coal dust deposits, for example, contain carbon, which can cause galvanic corrosion, and sulfur compounds, which attack steel. Bird droppings contain acids that damage steel members and protective coatings.” [1]

2.3.5 UNDERFILM CORROSION

DEFINITION

“Underwhelm corrosion is a type of crevice corrosion that occurs beneath paint. It usually begins where the paint has been physically damaged or at defects in the paint helm. This form of corrosion attacks the surface between the coating and the metal causing the paint to deboned. A special type of underwhelm corrosion known as filiform corrosion occurs in the form of threadlike filaments. Filiform corrosion occurs in high humidity conditions. Under film corrosion can be classified visually.” **[1]**

OCCURRENCE

“Under film corrosion starts at locations where there are breaks in the paint. It can occur anywhere on a structure and is seen as cracking, blistering, or peeling of the paint helm. Probing of the coating at damaged areas to determine if coating disbandment has occurred will often reveal that a much larger area of metal has been corroded than detectable by examining the painted surface.” [1]

2.3.6 PITTING

DEFINITION

“Pitting is localized corrosion attack which causes the formation of deep, sometimes narrow, penetrations into steel surfaces. Its formation occurs where there are chemical or physical changes in the metal such as imperfections in the metallurgy of steel, at paint protection flaws, or, most commonly, under deposits of foreign material. Pitting can act as a stress raiser and cause failure by cracking. Pitting can be identified with the naked eye.” **[1]**

OCCURRENCE

"Pitting is commonly found where debris of any type harbors moisture on a surface, such as deposits of dirt, trash, or bird excrement. Pitting commonly is found where the paint protection is scratched, nicked from flying debris from vehicles, or at imperfections in the application of the paint. Deposits of minute salt particles in coastal regions or where deicing salt is used can lead to extensive pitting. Pitting frequently takes place under deposits of corrosion product such as pack rust." [1]

2.3.7 INTERGRANULAR CORROSION

DEFINITION

"Intergranular corrosion is a corrosion attack of the boundaries between the metal grains. After the grain boundaries deteriorate, the grains fall out and the metal disintegrates. While the effects of intergranular corrosion are visible to the naked eye, a precise diagnosis requires supplementary examination." [1]

OCCURRENCE

"The most common form of intergranular corrosion on bridges is weld decay." [1]

2.3.8 WELD DECAY

DEFINITION

"Weld decay is the localized deterioration either of weld metal or base metal due to a decrease in corrosion resistance caused when the heat of welding alters the granular structure of the steel. This intergranular corrosion appears as a band of corrosion parallel to the weld. Weld decay usually requires supplemental examination to confirm its presence." [1]

OCCURRENCE

"Weld decay is not a common form of corrosion on bridges that have been properly welded under shop-controlled conditions during fabrication. Its occurrence is more likely to be found adjacent to field welds applied without proper control of heat. Paint applied over field welds may be of lower quality than shop paint, contributing to weld decay. It occurs more frequently in association with thin steels, stainless steels, and alloy steels, but can sometimes be found in structural carbon steels." [1]

2.3.9 SELECTIVE LEACHING

DEFINITION

"Selective leaching (sometimes referred to as de allowing) is the dissolution of one component of an alloy. This can result in changes in its mechanical properties. The identification of de alloying may require microscopic examination." [1]

OCCURRENCE

"Selective leaching is not commonly found on steel bridges. An example of such corrosion may be occasionally found on bronze (copper-zinc-tin alloy) bearings where the zinc may leach from alloy. Stagnant conditions in confined areas will favor its formation." **[1]**

2.3.10 EROSION CORROSION

DEFINITION

"Erosion corrosion is an attack on a metal caused by the flow of fluid over its surface with sufficient velocity to remove adhering surface corrosion product. Erosion corrosion, as it typically relates to bridges, is in the form of particle erosion, where particles in fluid abrade the metal surface, wearing away the surface coating on protective corrosion products. This allows corrosion to continually attack bare metal, and speeds the rate of attack. Erosion particle corrosion is analogous to water blasting with a grit. The identification of erosion corrosion may require microscopic inspection." [1]

OCCURRENCE

"Erosion particle corrosion is not a common form of corrosion on steel bridges but can be dangerous when streams carry particulate matter that erodes steel piling. This can go undetected under water." [1]

2.3.11 FRETTING CORROSION

DEFINITION

"Fretting corrosion is caused by relative motion of two surfaces in close contact under load. Fretting involves the rubbing contact 10 of non-lubricated surfaces where surface oxidation forms, is broken, and reforms, causing abrasion of the surfaces by oxide and debris. Fretting corrosion cannot be positively identified with the naked eye." [1]

OCCURRENCE

"On steel bridges fretting can be observed at stringer relief joints and at stringer ends' having sliding contact surfaces where slight stringer movement occurs, it may also be found at locations where bridge components vibrate." [1]

2.3.12 STRESS CORROSION

DEFINITION

"Stress corrosion cracking is cracking caused by the simultaneous occurrence of tensile stress (either residual or applied) and a corrosive environment. Corrosion causes the initiation of discontinuities in the metal acting as stress raisers that lead to cracks. The cracks may be either intergranular (around grains) or trans granular (across grains), but normally occur perpendicular to the member stress. Depending on the type of steel and the corrosive environment, the crack may be as simple as a single crack, but could have multiple branches. Stress corrosion cracking appears as a brittle fracture in an otherwise ductile metal. Upon microscopic examination, the corrosion product can be found in the cracks. The adjacent metal surface generally does not show evidence of any damage. Stress corrosion cracking requires microscopic inspection for identification." [1]

OCCURRENCE

"Stress corrosion cracking can occur in bridges under adverse environmental conditions, such as found in industrial areas or in marine environments. An example of stress corrosion cracking was observed in a structure in a corrosive environment where high-strength bolts failed while the

connected members showed no indications of corrosion. The bolts, being tensioned to the proof load (near yield point), developed cracks perpendicular to the applied load reducing the bolt cross-section area until the bolt failed. Stress corrosion cracking has also been observed on wires and strands in the main cables of suspension bridges." [1]

2.3.13 CORROSION FATIGUE

DEFINITION

"Corrosion fatigue is a fatigue-type cracking of metal caused by repeated or fluctuating applied stresses in a corrosive environment. It causes the reduction of fatigue life when the affected member is exposed to a corrosive environment compared to its life in a noncorrosive environment' The mechanism of corrosion fatigue is analogous to stress corrosion cracking, with corrosion creating stress concentrations which cause crack initiation. The damage appears to occur only during the tensile stress portion of the fatigue stress cycle. Corrosion fatigue must be verified by microscopic examination." [1]

OCCURRENCE

"The occurrence of corrosion fatigue on steel bridges is limited to fatigue-sensitive members in a corrosive environment. The distinction between corrosion fatigue and normal fatigue is determined by the presence or absence of corrosion." [1]

2.2 PICTURES FOR TYPES OF CORROSION OCCURS IN THE BRIDGE ELEMENTS

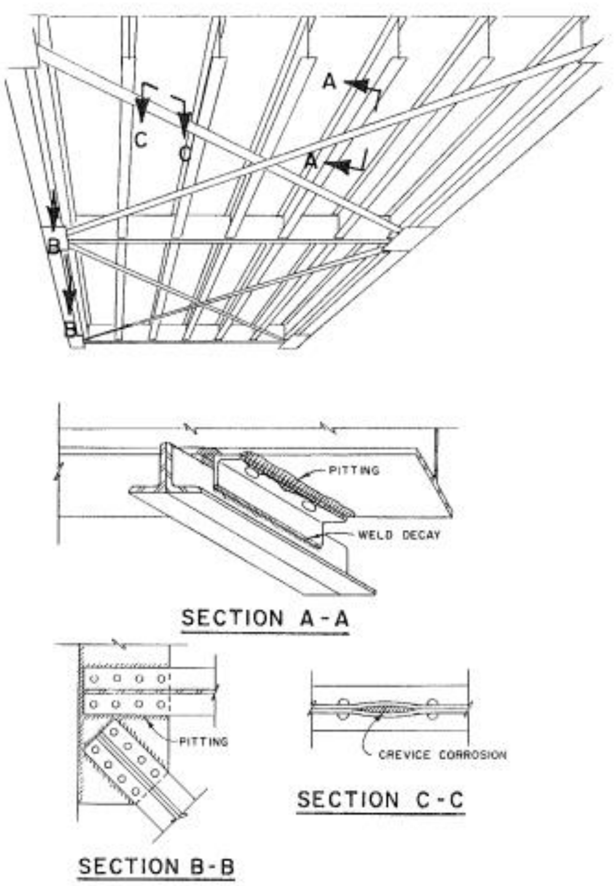


Fig.5 Deposit, crevice, pitting and weld decay attack the bottom beam [2]

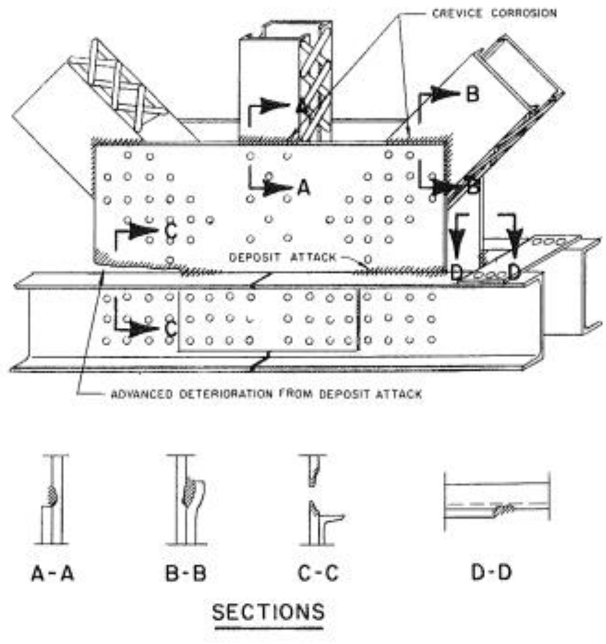


Fig.6 Deposit, crevice attack the connection plate [2]

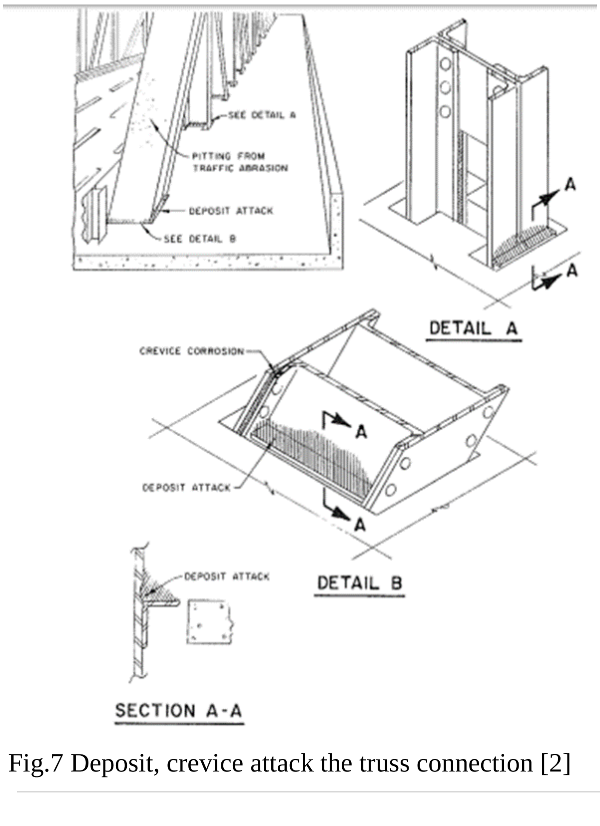


Fig.7 Deposit, crevice attack the truss connection [2]

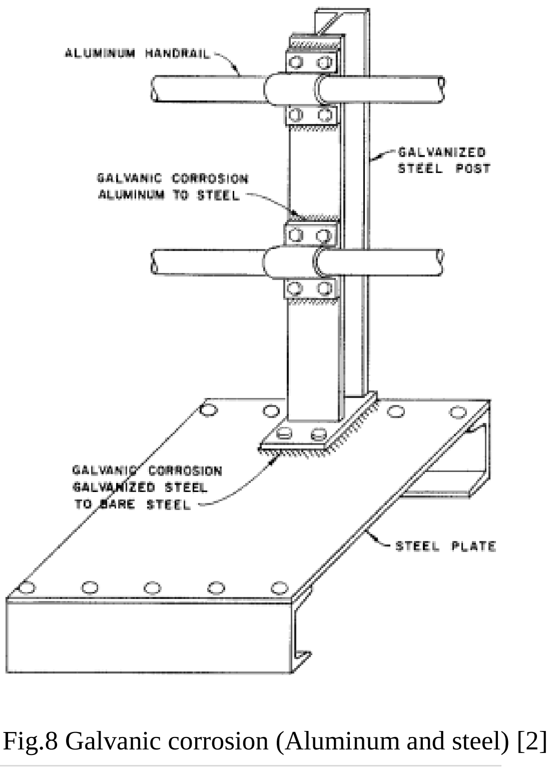


Fig.8 Galvanic corrosion (Aluminum and steel) [2]

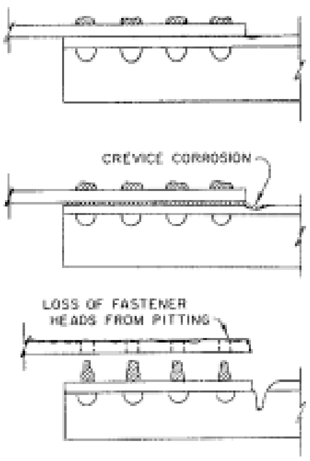


Fig.9 Loss of fastener head due to pitting [2]

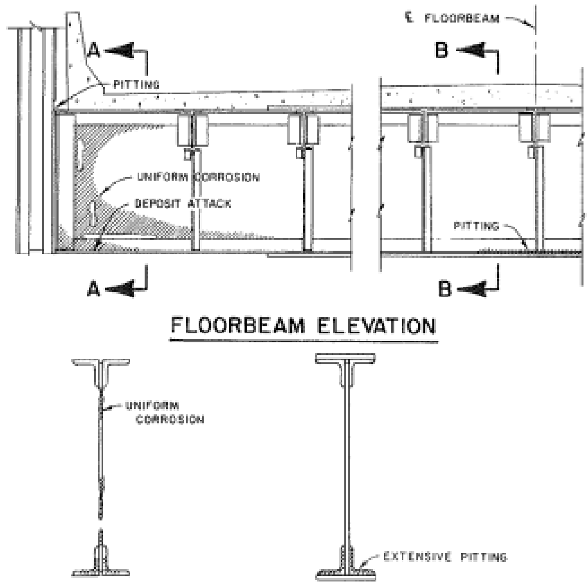


Fig.10 Deposit, uniform and pitting corrosion attack the floor beam [2]

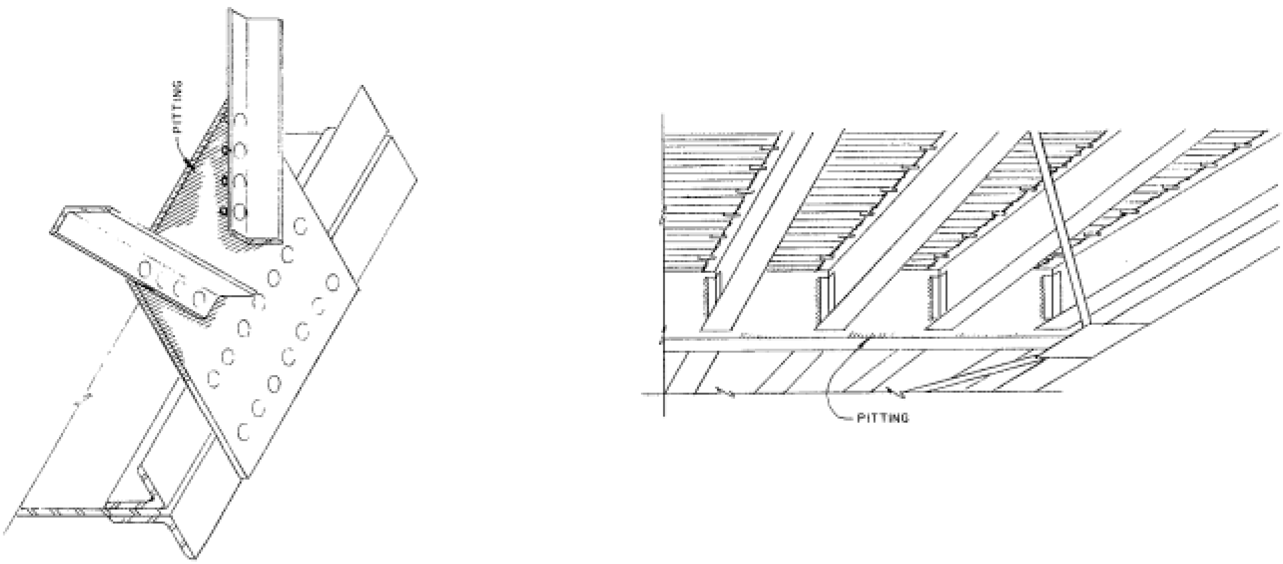


Fig.11 Pitting corrosion attack the girder lateral connection and stringer bottom [2]

2.4 TYPES AND TECHNIQUES OF CORROSION INSPECTION

2.4.1INTRODUCTION

“The corrosion inspection of steel bridges is similar to the well-established maintenance or safety inspection of bridges generally performed biannually under FHWA guidelines and reported on SI&A forms. The only difference is that the entire emphasis of the inspection is on corrosion. The corrosion inspection is not intended to alter the existing National Bridge Inspection Standards but rather to supplement them. The corrosion inspection identifies types of corrosion, records their effects, and appraises corrosion conditions. It is a specialized inspection involving bridge inspectors with added training and knowledge of corrosion.” [2]

2.4.2 LEVEL I. CURSORY INSPECTION

“The cursory inspection provides an overview of corrosion conditions without detailed examination of deficient areas or the use of sophisticated tools and equipment, by using visual observation and experience to evaluate the conditions. The cursory inspection answers such basic questions as: (1) Is extensive corrosion present? (Without actually measuring metalwork losses') (2) Is corrosion global (found throughout the entire bridge) or localized? (3) Has corrosion caused misalignment of parts, shifted bridge components, or frozen members intended to move? (Without actually measuring the amount of displacement or fixity.” [2]

GENERAL INSPECTION

“The general corrosion inspection is used for a Level I evaluation. The inspection is a "hands-on" approach in which bridge members that are accessible without the need for specialized equipment are climbed and inspected, and where random or spot measurements are taken to quantify the extent of metalwork losses. Both general and worst case conditions are checked. A combination of estimating and measuring is used for determining the extent of corrosion damage.” [2]

DETAILED INSPECTION

“The detailed inspection is an in-depth inspection covering all corrosion aspects of all bridge elements. If necessary, special access-gaining equipment is used to put the inspector in a "hands-on" position to closely observe each member and make detailed measurements of all metalwork losses. Metalwork surface cleaning is performed, as required, to make accurate surface measurements and precise determination of metalwork losses.” [2]

INSPECTION PERSONNEL

“The inspection personnel performing corrosion inspections should have the same minimum qualifications as required for the bridge maintenance inspection program. These qualifications include being in good physical condition; a minimum of a high school education; training in bridge maintenance inspection plus added training in corrosion; the physical ability to climb structural steel without difficulty; and the skills needed to inspect, sketch, report, photograph, and measure details. The qualifications of an inspector should be matched to the level of corrosion evaluation required. For Level II evaluations, corrosion experts may be included in the inspection team.” [2]

2.5 QUANTIFICATION OF CORROSION DAMAGE

"In order to quantify the corrosion damage measured and reported by the field inspector, the following parameters are used throughout this study:

l. Percentage of section loss, % loss-The percentage of section loss defines the amount of metal loss at a given location on a bridge member. It relates the amount of section loss to the original section of the member:

$$\% \text{ loss} = \left(1 - \frac{Ad}{A}\right) 100$$

Where, "A" is the original cross-sectional area and Ad is the reduced section area. The term "percentage loss" is familiar to bridge inspectors and is frequently used in inspection reports.

2. Loss coefficient, Q-The loss coefficient, Q, also describes the amount of metal loss at a given location along a member. It is deformed as the ratio of original section area, A, to the reduced section area:

$$Q = \frac{A}{Ad} = \frac{100}{100 - \% \text{ loss}}$$

The advantage of this parameter is that it is linearly proportional to the increase in stress in an axially loaded bar. Thus, it facilitates comparison of the effects of material loss in various members to the simple case of loss of section in an axially loaded bar.

3. Length of loss (l) the length of loss, (l) defines the extent of loss along a member. It is usually assumed that along the length, (l) the percentage of section loss is constant.

4. Transition from reduced to full section: The type of transition from reduced to full section has to be defined as well. The transition can be abrupt or gradual, at a given rate or have a given radius" [2]

2.6 EVALUATION CRITERIA FOR CORROSION AFFECTED BRIDGE

2.6.1 GENERAL

"*The conditions created by corrosion can result in various modes of failure that are not necessarily those that controlled the original design of the bridge. In order to evaluate the condition of a bridge affected by corrosion, several criteria need to be considered: strength, deformation, stability, fatigue, fracture, redundancy, and criticality of member or detail. For each condition, the applicable criteria must be identified and addressed. It can be beneficial to relate these criteria to the original design criteria of the bridge, if available. A brief description of the evaluation criteria is given below.*" [2]

2.6.2 STRENGTH CRITERIA

"The residual strength of a deteriorated member may be determined by using a service load approach or a load factor approach." [2]

2.6.3 SERVICE LOAD APPROACH.

"*The service load or the allowable stress approach uses the attainment of first yielding as a basis for defining the limit state of structure or a member. Safety is ensured by limiting stresses to allowable values, which are below the elastic limit of steel. This facilitates the use of a linear elastic method of analysis. In some cases, the service load approach recognizes the possibility that yielding due to stress concentrations or residual stresses may take place at service load levels without resulting in unrestrained plastic flow and section failure. For example, in axially loaded members, uniform stress distribution is assumed in spite of the possible existence of bolt or rivet holes, residual stresses, or other stress concentrations. The service load approach has been used for the design of most of the existing steel bridges and it still is the most common approach to bridge design and rating. It is needed for serviceability, fatigue and fracture evaluations, even if a load factor approach is used.*" [2]

2.6.4 LOAD FACTOR APPROACH.

"Strength or a load factor approach uses the ultimate strength of a member as a limit state. Safety is ensured by limiting the load to a level below that which would cause failure of a member or collapse of the structure. A load factor approach can recognize reserves of strength beyond first yield that may result from stress redistributions. Current practice is to use a linear elastic method of analysis to determine member loads and then use a strength approach at the member level. Strength type approaches are becoming more and more accepted in bridge design and rating." [2]

2.6.5 DEFORMATION CRITERION

"The deformation criterion is primarily related to the serviceability of the structure. Loss of material due to corrosion may lower the stiffness of the structure and result in unacceptable deflections and deformations. When the deformations become inelastic, the strength of the structure may also be affected. The approach used to verify the deformation criterion is the service load approach with an elastic method of analysis. Corrosion can reduce the stiffness of members and thus result in increased deformations." [2]

2.6.6 STABILITY CRITERION

"The stability criterion includes local instability, member instability, and structural instability. Instability can initiate in the elastic or the plastic range' in many cases the stability criterion will control the design of a member or structure. Stability is ensured by modifying allowable stresses if a service load approach is used or by modifying the ultimate strength criteria' Corrosion can induce eccentricities and reduce section properties such as moment of inertia and radius of gyration and thus lower the resistance to local or overall buckling." [2]

2.6.7 FATIGUE CRITERION

"The *fatigue criterion addresses the behavior of the structure under repeated loading. It has to ensure that no fatigue cracks develop during the expected life of the structure. Fatigue cracks are generally initiated in regions of maximum tensile stresses at points of stress concentration such as holes, notches, or other imperfections and discontinuities. The technique used to verify the fatigue criterion is the service load approach, with an elastic method of analysis. Some of the conditions created by corrosion can affect the fatigue resistance of a structure. Uniform corrosion results in surface roughness which corresponds to localized stress raisers on the surface. Localized corrosion can create eccentricities, holes, and other abrupt discontinuities which can result in a reduced fatigue resistance. Corrosion fatigue, which may occur when the structure is exposed to a corrosive environment, can also reduce the fatigue resistance. Fretting corrosion can initiate cracks and thus adversely affect fatigue performance. In general, the effect of corrosion on a member will depend on its original condition. The effect on a rolled member will be more significant than the effect on a member with poor fatigue details such as weldments or rivet holes."* [2]

2.6.8 FRACTURE CRITERION

"The fracture criterion addresses the possibility of a member fracture. The fracture can be either brittle or ductile. Brittle fracture occurs without prior yielding, while ductile fracture is generally preceded by some local plastic deformation' Certain service conditions such as low temperature, impact loading of members with severe discontinuities and conditions of high constraint that restrict the capacity for local yielding greatly affect the susceptibility to brittle fracture. Fracture also occurs at discontinuities that grow to a critical size as a result of fatigue or stress corrosion. The fracture criterion is based on service load conditions." [2]

2.6.9 REDUNDANCY

"*A redundant structure is a structure where failure of a single member cannot lead to total collapse' Redundancy is related to the ability of a structure to redistribute loads after one or more of its components fail. Evaluation of structural redundancy requires a good understanding of the behavior of the structure and of the importance of the damaged member. Redundancy is becoming increasingly accepted as a criterion in the design of new bridges and in the evaluation of existing bridges' The effects of corrosion on a highly redundant bridge structure will not be as significant as on a bridge structure with very little redundancy. Many existing bridges are unintentionally redundant' There are many actual cases in which failure of a bridge member or connection, even though considered critical in the original design, did not result in total bridge collapse. There are cases in which one channel of a two channel built-up bottom chord in a truss bridge failed and the bridge still carried dead and live load, cases in which the whole* *bottom chord of a truss failed and the floor system carried the load, and cases in which a girder of a two girder continuous bridge failed and the bridge did not collapse in other cases, however, failure of one member, such as the end post of a truss, or a connection, such as a pin 'hanger or eye bar joint, caused a bridge collapse."* [2]

2.6.10 CRITICALITY OF MEMBER OR DETAIL

"*The criticality of a bridge member or detail is related to the consequence of failure of that member or detail. In some cases, failure of a member has little effect on the structural integrity of the bridge, while in other cases it can cause sudden collapse. The criticality of a bridge member is determined by the following factors: location and function, redundancy, and mode of failure. Not all members of a bridge control its load-carrying capacity. For example, the posts in a Warren through-truss provide bracing to the top chord but do not carry any primary dead load or live load. The top chord members and the bottom chord members, however, directly carry the compression and the tension loads from dead load and live load. The importance of web member's increases from mid-span to the end of the span. Not all members of a bridge that control its load-carrying capacity are equally critical. If a member is highly redundant (made of several parallel elements) and is not required for stability, it will be able to sustain failure of one of its elements without serious consequences. Damage of a member which is not internally or structurally redundant, however, can result in the collapse of a single load-path structure. The mode of failure a member is likely to undergo also affects its criticality. Slow deterioration of a bending member is not as critical as sudden failure of a tension member due to fracture or sudden failure of a compression member due to instability. Thus, the most critical members in a bridge structure are non-redundant members which control the load-carrying capacity and whose failure would be expected to result in a sudden bridge collapse. They include tension members defined by AASHTO as fracture critical members (FCM's), and compression members which can fail through instability. These members should receive a more rigorous evaluation*." [2]

2.6.11 CHANGES IN DESIGN CRITERIA

"*The developments in methods of analysis, the continuing bridge-related research, and the experience accumulated over the years have led to changes in the criteria for bridge design. It may prove useful to relate the criteria used for evaluating corrosion effects on an existing bridge to the original design criteria of that bridge. In most cases, it will be found that the original design criteria regarding the assessment of the resistance of bridge members are more conservative. The evaluation criteria used may be related to the original design criteria through a code factor, CF, defined as the ratio of the capacity of a member calculated based on the present criteria, to the capacity of that member calculated based on the original design criteria. A member code factor, CF, larger than 1.0 would indicate that the member has some capacity in excess of that assumed in the original design. When taking into account the changes that occurred in the design criteria related to member resistance, the changes in the loading of bridges that occurred over the years should also be considered if a load-carrying capacity evaluation is made."* [2]

2.6.12 FACTORS OF SAFETY

"*The factors of safety used affect the evaluation of the residual capacity and the remaining load-carrying capacity of a bridge member or structure. They have little effect on the evaluation of residual capacity factors. When calculating residual capacity factors, the same* *safety factors will appear in both the numerator and the denominator and in most cases they will cancel out. In the service load approach, factors of safety are included in the allowable stresses specified by AASHTO. In the load factor approach, factors of safety are included as load and capacity reduction factors and they account for the uncertainties of loadings and structural response. The ultimate cross-sectional capacity is assumed for determining member resistance. In the load and resistance factor approach, factors of safety are included in the form of load factors and resistance factors. They are probability-based and account for uncertainties in both the loadings and the structural resistance. The factors of safety used for bridge evaluation are usually smaller than those used in design. AASHTO uses an inventory and operating level for bridge rating, and the Ontario Highway Bridge Design Code (6. 12), permits a reduction in the live load factor from l.40 to 1.25 for the evaluation of existing bridges*." [2]

2.7 BRIDGE SAFETY FACTOR

"The bridge safety factor is a numeric value calculated according to the formula, which shows if a particular bridge on a particular road with a certain traffic volume is safe, reliable and fit for use. If the value of the safety factor is larger than one, the load carrying capacity of the bridge is sufficient. If the value is lower than one, the load carrying capacity of the bridge is in sufficient. That is a signal that the volume of heavy weight vehicles has to be limited, the bridge has to be repaired or demolished, and a new bridge has to be built." [3]

$$\boxed{RF = \frac{\phi \, . \, R_d - \gamma_G \cdot G_N}{\gamma_Q \, . \, G_Q \cdot \mu_{din}}} \qquad \text{equation 1}$$

$$\boxed{\phi = \frac{1}{e^{\alpha R}}} \qquad \text{equation 2}$$

Where,

RF is the bridge safety factor;

Φ is the index weakening the strength of the bridge superstructure equation 2;

R_d the load carrying capacity of a bridge superstructure;

γ_G is the reliability coefficient of dead loads;

G_N is the effect of dead loads;

γ_Q is the coefficient of the effect of traffic volume and live loads, from table 2;

G_Q is the effect of live loads;

μ_{din} is the dynamic coefficient of the bridge superstructure;

$e^{\alpha R}$ is the wear factor of the bridge load carrying structures presented in table 1;

Table 1. Values of the wear factor of the bridge load carrying structures

Condition evaluation in points	Wear factor of load carrying structures, αR	Values of the index weakening the strength of the bridge load carrying structures, ϕ
1	0.35	0.71
2	0.25	0.78
3	0.20	0.82
4	0.10	0.90
5	0.05	0.95

Table 2. Coefficient values of the effect of live loads

Volume of heavyweight vehicles, vehicles/day	Coefficient of the effect of live loads, γ_Q
< 250	1.50
> 250 < 1000	1.60
> 1000 < 5000	1.70
> 5000	1.80

3 GENERAL DATA OF THE BRIDGE

Table 3. General data of the bridge

Path name	Road No.	Bridge index	Km	Bridge length (m)	Building of the year
Kaunas-Jurbarkas-Silute-Klaipeda	141	KLKL020T1921P079MIN	210.14	75.10	1921

Bridge type	Steel arch bridge		
Span overlay lengths (m)	The first	The second	The total length of
	36.72	36.72	73.44
Design	German design standards **DIN1072**		
The construction of the bridge overlay	Two metal bolted truss with vertical member and complex strut. Between the time the cushioning of the truss on the transversal and longitudinal beams across the bridge to put stringer. Stringers complete the hewn stones.		

Fig.12 Longitudinal Bridge view

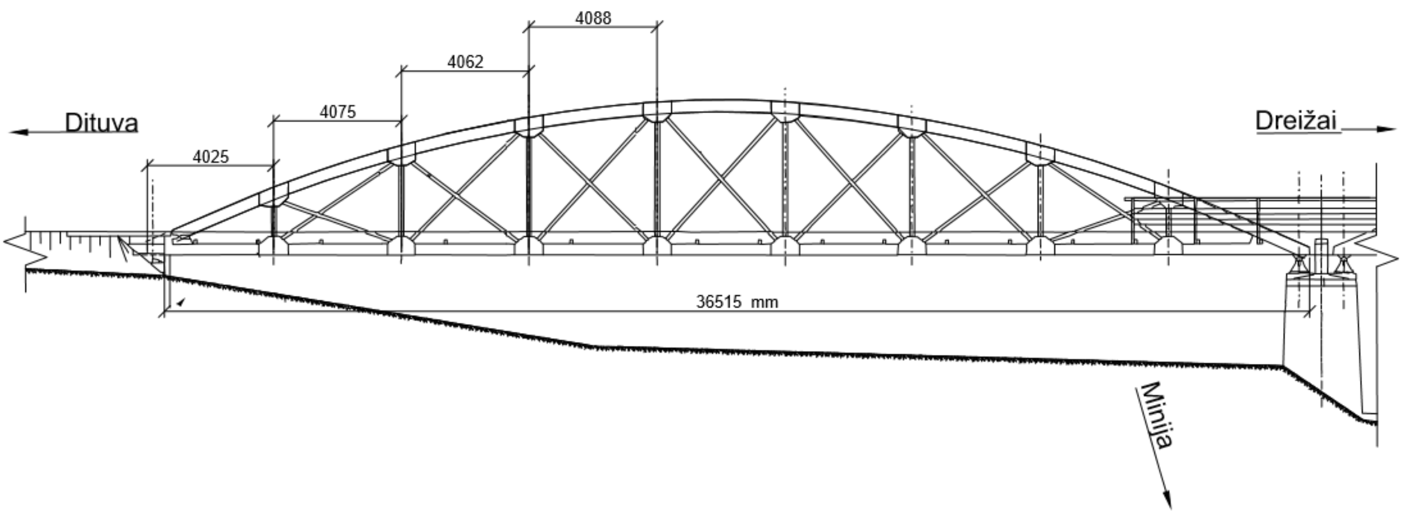


Fig.13 Bridge elevation view

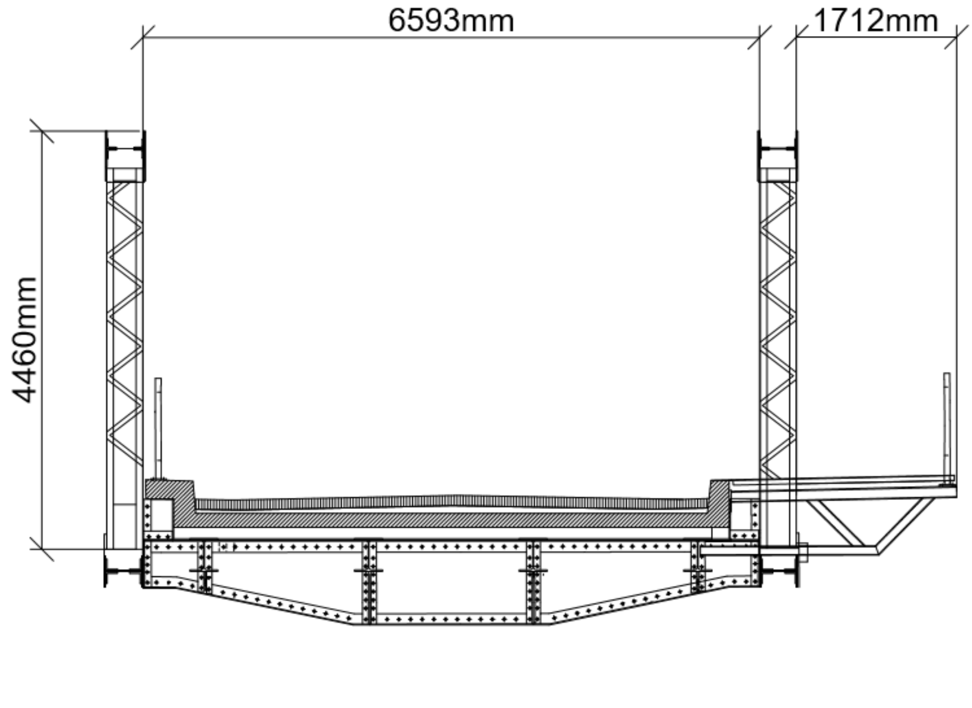


Fig.14 Bridge section view

4 CORROSION OBSERVATION PICTURES

Fig.15 Bridge up stream

Fig.16 Bridge longitudinal

Fig.17 Bottom end stringers corrosion

Fig 18 Lateral and cross beams corrosion

Fig.19 Bottom stringer significant corrosion


Fig.20 Bottom middle stringer and beams


Fig.21 Bottom end stringer and beam significant corrosion

Fig.22 Bottom end corrosion damage

Fig.23 Stringer crack due to the corrosion

Fig.24 Broken connection upper first support

5 CORROSION SURVEY RESULTS

OVER ALL CONDITION (VISUAL INSPECTION).

i. All the cushioning of the whole stringers corroded, significantly reduced cross sections, metal-crazed.

ii. Transverse and longitudinal beams greatly damaged by corrosion.

iii. The protective paint coating depreciated truss elements.

iv. Outdated structure and fully depreciated items. The asphalt coating cracked, too narrow (5.26 m wide), the road and bridge junctions slit, and Waterproofing is not available. Water percolating through the cushioning.

6 CALCULATION OF THE PERCENTAGE OF LOSS

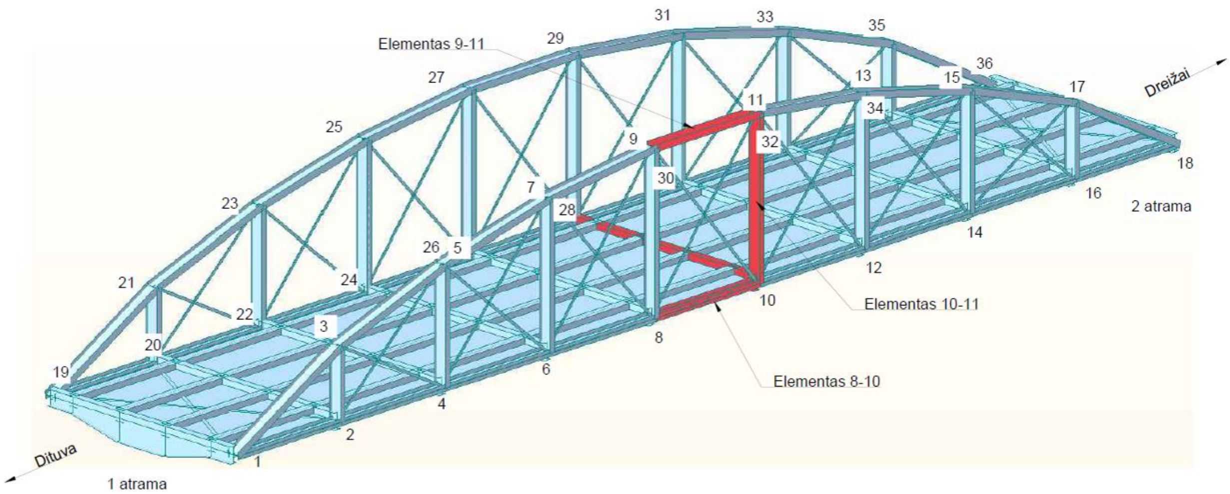


Fig.25 Elements carrying maximum load in the bridge

6.1 MATERIAL LOSSES

6.1.1 UPPER BAND

According to the corrosion observation the upper band average thickness losses is 3 mm in the element cross-section due to the uniform corrosion.

Before corrosion Area (A) = 0.01562 m^2

After corrosion Area (AD) = 0.014058 m^2

Percentage of loss % = $(1-AD/A)$ 100 = 10%

6.1.2 CROSS BEAM

According to the corrosion observation the cross beam average thickness losses is 6 mm in the element cross-section due to the **river water**.

Before corrosion Area (A) = 0.000486 m^2

After corrosion Area (AD) = 0.000388 m^2

Percentage of loss % = $(1-AD/A)$ 100 = 20%

6.1.3 BOTTOM TRUSS BEAM

According to the corrosion observation the bottom truss beam average thickness losses is 5 mm in the element cross-section due to the **river water**.

Before corrosion Area (A) = 0.0133302 m^2

After corrosion Area (AD) = 0.01133067 m^2

$$\text{Percentage of loss \%} = (1-AD/A)\ 100 = 15\%$$

6.1.4 STRUT

According to the corrosion observation the strut average thickness losses is 3 mm in the element cross-section due to the **uniform corrosion.**

Before corrosion Area (A) = 0.001748 m^2

After corrosion Area (AD) = 0.0016606 m^2

$$\text{Percentage of loss \%} = (1-AD/A)\ 100 = 5\%$$

7 VEHICLE LOAD WITH INFLUENCE LINE

7.1 UPPER BAND.

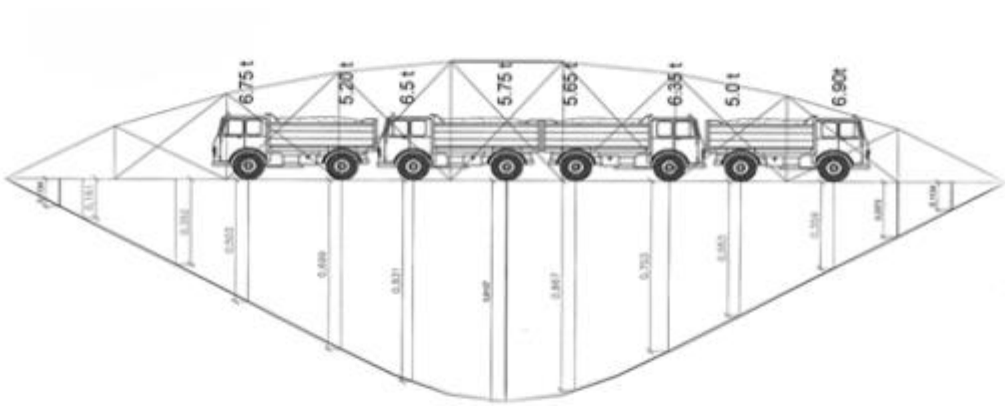


Fig.26 Upper band vehicle load influence line

Weight of the member (W2) = 0.00003834 m^3

Area of the member (A1) = 0.01562 m^2

Total load (N1) = {(66.21 x 0.503) + (51.012 x 0.699) + (63.765 x 0.831) + (56.40 x 0.9137) + (55.42 x 0.867) + (62.29 x 0.703) + (49.05 x 0.553) + (67.68 x 0.359)}

$$\boxed{\mathbf{N1 = 316.74\ kN}}$$

$$\textbf{Stress} (\sigma) = \frac{N1}{A1} = \frac{\mathbf{316.74}}{\mathbf{0.01562}} = 20.27\ \text{N/mm}^2$$

7.2 STRUT

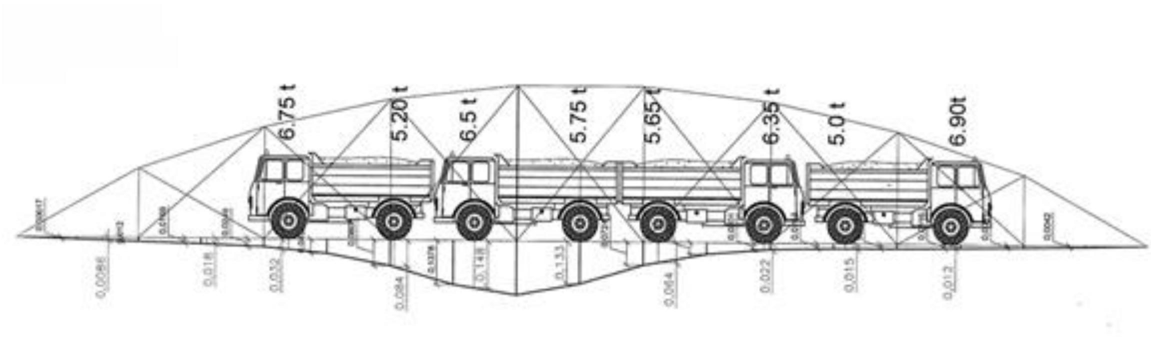


Fig.27 Strut vehicle load influence line

Weight of the member (W2) = 0.00060752 m^3

Area of the member (A2) = 0.001748 m^2

Total load (N2) = {(66.21 x 0.032) + (51.012 x 0.084) + (63.765 x 0.148) + (56.40 x 0.133) + (55.42 x 0.064) + (62.29 x 0.022) + (49.05 x 0.015) + (67.68 x 0.012)}

$$\boxed{\mathbf{N2 = 29.80\ kN}}$$

$$\textbf{Stress } (\sigma) = \frac{N2}{A2} = \frac{29.80}{0.001748} = 17.052 \text{ N/mm}^2$$

7.3 BOTTOM TRUSS BEAM

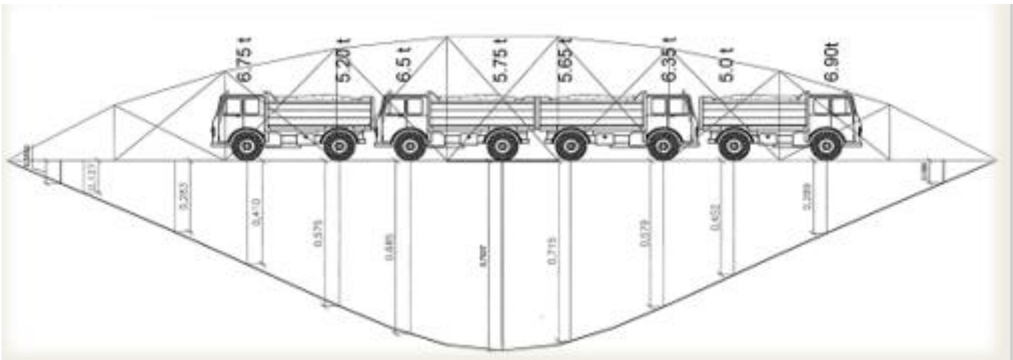


Fig.28 Bottom truss beam vehicle load influence line

Weight of the member (W3) = 0.00050605 m^3

Area of the member (A3) = 0.0133302 m^2

Total load (N2) = {(66.21 x 0.410) + (51.012 x 0.575) + (63.765 x 0.685) + (56.40 x 0.754) + (55.42 x 0.72) + (62.29 x 0.60) + (49.05 x 0.452) + (67.68 x 0.30)}

$$\boxed{\mathbf{N2 = 262.43\ kN}}$$

$$\textbf{Stress } (\sigma) = \frac{N3}{A3} = \frac{262.43}{0.0133302} = 19.687 \text{ N/mm}^2$$

7.4 CROSS BEAM

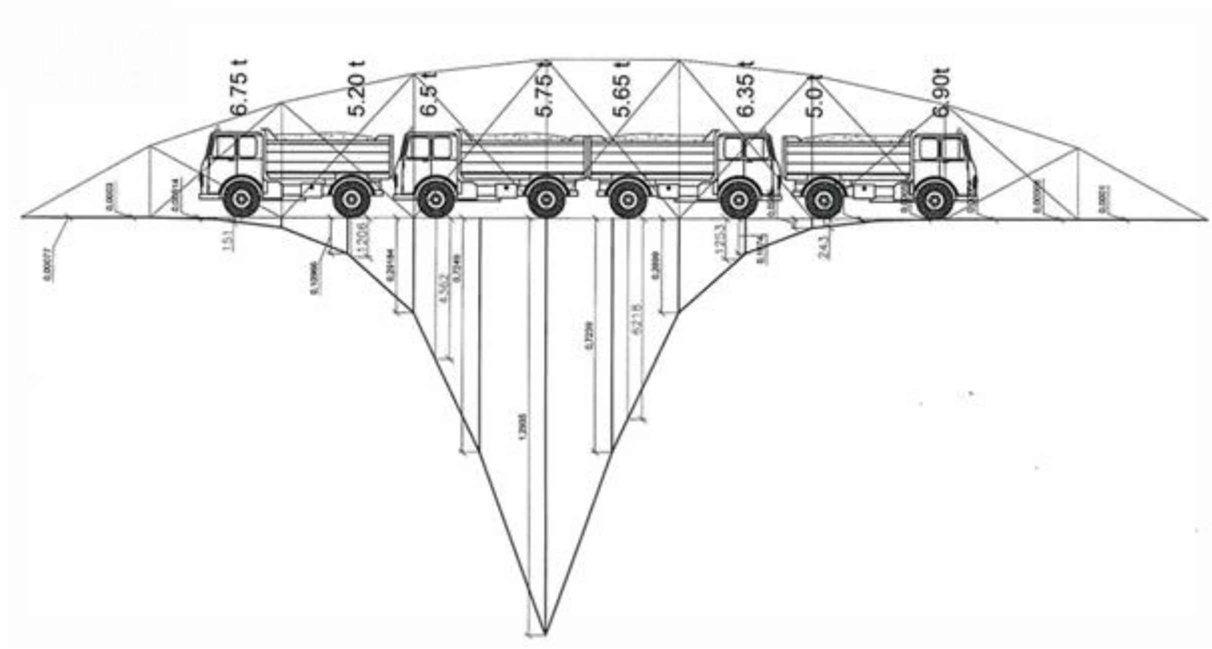


Fig.29 Cross beam vehicle load influence line

Weight of the member (W4) = 0.0042119 m^3

Area of the member (A4) = 0.000486 m^2

Total load (N4) = {(66.21 x 0.00514) + (51.012 x 0.1206) + (63.765 x 0.4362) + (56.40 x 1.30) + (55.42 x 0.6218) + (62.29 x .12) + (49.05 x 0.243) + (67.68 x 0.0006)}

$$\boxed{\mathbf{N4 = 148\ kN}}$$

$$\textbf{Stress } (\boldsymbol{\sigma}) = \frac{N3}{A3} = \frac{148}{0.000486} = 304.526 \text{ N/mm}^2$$

8 STRENGTH COMPARISON

Table.4 Bridge elements strength comparison

Sl.No	Member	Total load (KN)	Cross sectional area (A)m^2	Before corrosion (Ϭ)MPa	Cross sectional area (A)m^2	After corrosion (Ϭ)MPa	Material strength fyd MPa
1	Upper band	316.74	0.01562	20.27	0.014058	22.530	240
2	Cross beam	160	0.000486	176.2	0.000388	218.4	240
3	Bottom truss beam	262.43	0.0133302	19.687	0.01133067	23.161	240
4	Strut	29.80	0.001748	17.052	0.0016606	17.945	240

8.1 DEFLECTION

To calculate the maximum deflection of the cross beam due to the maximum load.

<u>BEFORE CORROSION:</u>

Material = S355

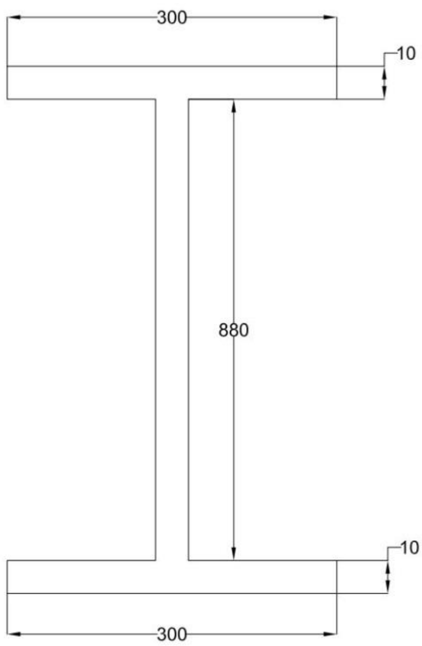
Young's modulus (E) = 210 N/mm^2

Moment of inertia (I) = 1756093333.34 mm^4

Length (L) = 6600 mm

Area (A) = 14800 mm^2

Total load (P) = 25.37 kN/m



Deflection (Y) $= \dfrac{wL^3}{48EI} = \dfrac{25.37 \times 6600^{\,3}}{48 \times 210 \times 17.57 \times 10^{\,8}}$

$\boxed{Y = 0.4118\ \text{mm}}$

Fig.30 Cross beam original cross section

<u>AFTER CORROSION:</u>

Due to the deterioration of corrosion the top and bottom flange of the beam reduce 2 mm and 4 mm from its original thickness because of this changes the moment of inertia and Area of thee material is I = 1388752961 mm^4, Area = 13000 mm^2 and also assume that the elastic modulus of the material reduces 20% and 40% (E = 168 N/mm^2), (E = 126 N/mm^2) from its original strength (E = 210 N/mm^2), because of the corrosion and material of this bridge passed over 100 years.

Young's modulus (E) = 168 N/mm^2

Moment of Inertia (I) = 1388752961 mm^4

Deflection (Y) $= \dfrac{wL^3}{48EI} = \dfrac{25.37 \times 6600^{\ 3}}{48 \times 168 \times 13.88 \times 10^{\ 8}}$

$\boxed{\mathbf{Y = 0.651mm}}$

Young's modulus (E) = 126 N/mm^2

Moment of Inertia (I) = 1388752961mm^4

Deflection (Y) $= \dfrac{wL^3}{48EI} = \dfrac{25.37 \times 6600^{\ 3}}{48 \times 126 \times 13.88 \times 10^{\ 8}}$

$\boxed{\mathbf{Y = 0.868\ mm}}$

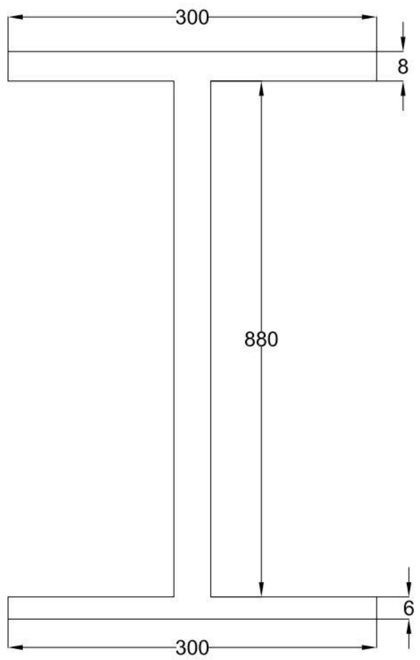


Fig.31 Cross beam corroded section

8.2 TRUSS DEFORMATION

The truss deformation measured by using the electronic device. It is measured due to symmetrical load and unsymmetrical load.

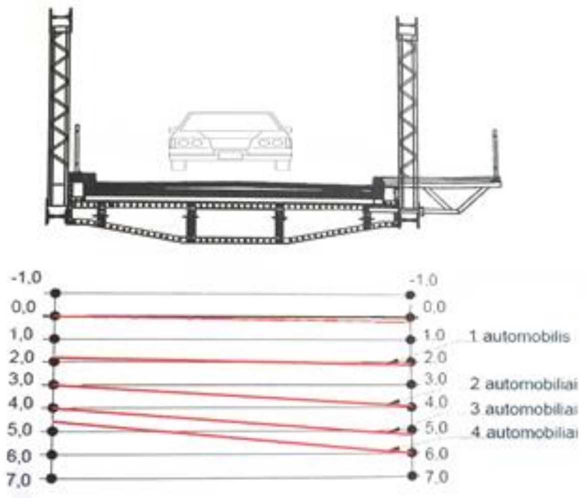


Fig.32 Truss deformation due to symmetrical load

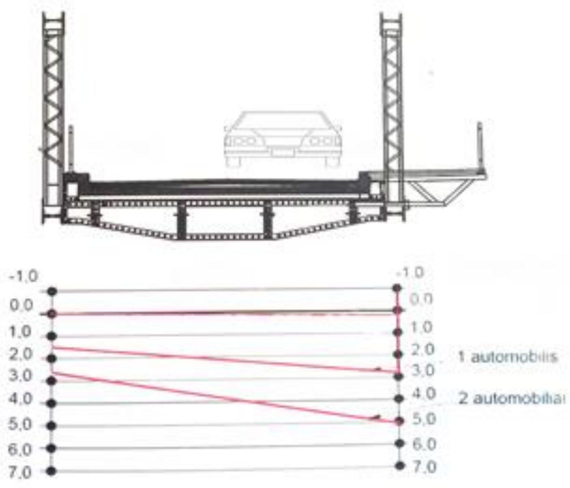


Fig.33 Truss deformation due to unsymmetrical load

9 BRIDGE SAFETY FACTOR

The safety factor for the bridge was calculated according to the strength of the superstructure for the most loaded element **9-11**.

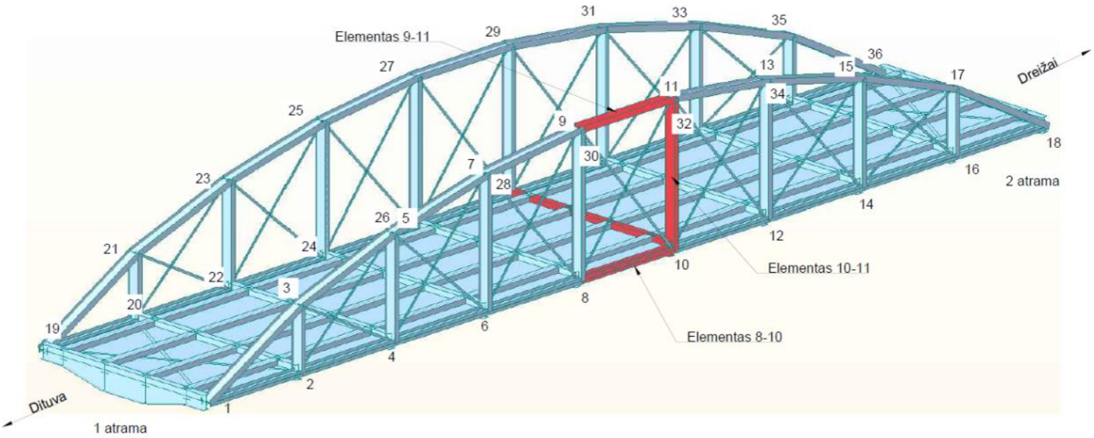


Fig. 34 Calculation scheme for the superstructure of the bridge over the river Minija with marked elements

Table 5. Key data for the calculation of the bridge safety factor.

Sl No	Description of the calculated value	Calculated value
1	Condition value of the bridge load carrying structures, points	4
2	Wear factor of the load carrying structures, αR	0.10
3	Index weakening the strength of the bridge load carrying structures, ϕ	0.90
4	Dynamic coefficient of the bridge superstructure, μdin	1.17
5	Reliability coefficient of dead loads, γG	1.35
6	Coefficient of the effect of live loads, γQ	1.50
7	Effect of dead loads, Gn kNm	1400
8	Effect of live loads, GQ kNm	167
9	Stability of the heaviest loaded element of the bridge superstructure under compression, MN, kNm	2819
10	Bridge safety factor RF in relation to the strength of the superstructure calculated according to the formula	2.21

9.1 CALCULATION OF THE SAFETY FACTOR

1) Dynamic coefficient = 1.17 (Heavy weight vehicle was moving at 30km/h speed.
2) Reliability coefficient of dead loads = 1.35. (European standards)
3) Effect of live loads = 167kN (Element 9-11, when four-axle heavy weight vehicle appears at the most inconvenient point)
4) The greatest permissible load is 38t or 9.50t per axle.
5) The traffic volume of heavy weight vehicles is 60vehicles/day.
6) The coefficient of live loads = 1.50.

$$\text{RF} = \frac{\phi \, . \, N_N - \gamma_G \cdot G_N}{\gamma_Q \, . \, G_Q \cdot \mu_{din}}$$

$$= \frac{0.9 \times 2819 - 1.35 \times 1400}{1.5 \times 167 \times 1.17} = \frac{2537 - 1890}{293}$$

Safety factor (RF) = 2.21

9.2 IN ORDER TO FIND OUT HOW THE VALUES OF THE SAFETY FACTOR CHANGE AND WHICH FACTORS HAVE THE GREATEST INFLUENCE.

1. *“Increase in the dynamic coefficient;*
2. *Deteriorated condition of the superstructure by one point;*
3. *Deteriorated condition of the superstructure by one point and improved dynamic coefficient;*
4. *Deteriorated condition of the superstructure by one point, improved dynamic coefficient, higher traffic volume and heavyweight vehicles permissible mass exceeded by 10%*
5. *Deteriorated condition of the superstructure by two points.”* [3]

Table.6 Values of the safety factor while changing the main parameters

Sl No	Description of the calculated value	Existing situation	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5
1	Condition value of the bridge load carrying structures, points	4	4	3	3	3	2
2	Wear factor of the load carrying structures, αR	0.10	0.10	0.20	0.20	0.20	0.25
3	Index weakening the strength of the bridge load carrying structures, ϕ	0.90	0.90	0.82	0.82	0.82	0.78
4	Dynamic coefficient of the bridge superstructure, μdin	1.17	1.25	1.17	1.25	1.25	1.17
5	Reliability coefficient of dead loads, γG	1.35	1.35	1.35	1.35	1.35	1.35
6	Coefficient of the effect of live loads, γQ	1.50	1.50	1.50	1.50	1.50	1.50
7	Effect of dead loads, Gn kNm	1400	1400	1400	1400	1400	1400
8	Effect of live loads, GQ kNm	167	167	167	167	167	167
9	Stability of the heaviest loaded element of the bridge superstructure under compression, MN, kNm	2819	2819	2819	2819	2819	2819
10	Bridge safety factor RF in relation to the strength of the superstructure calculated according	2.21	2.07	1.44	1.35	1.26	1.05

CONCLUSION

The steel truss bridge evaluated results shows that the truss elements corrosion it is not make a major problem but the bottom of the bridge was deeply corroded by the river water and the appearance is very bad. The percentage of loss for the upper band is 10%, cross beam 20%, bottom truss beam 15% and strut 5% from its original cross sectional area. According to the strength calculation the stress increased due to the corrosion for the upper band is 20.27 MPa to 22.530 MPa, cross beam 176.2 MPa to 218.4 MPa, bottom truss beam 19.687 MPa to 23.161 MPa and strut 17.052 MPa to 17.945. It is revealed that the deterioration of the steel affects the strength of the steel.

The deflection of the cross beam results shows that due to the reduction of the top flange (2mm) and bottom flange (4mm) thickness of the beam affect to get more deflection. The assumption of the 20% and 40% reduction of the elastic strength shows that the deflection is increased (0.651mm and 0.868mm). This assumption is used to know the possible maximum deflection of the beam due to the maximum load where we could not able to test the sample of the material of the bridge, which is passed over many years. It is made for the safety purpose. The truss deformation due to the symmetrical and unsymmetrical load it is safe.

The bridge safety factor is an indicator for comparing load carrying capacities of all bridges based on the same criteria. The proposed scenarios showed that the most important criteria include the load carrying capacity of the condition value of the bridge structure; an important role is played by the dynamic coefficient and the effect of live loads.

The calculated bridge safety factor for this bridge is 2.21 according to the condition value, effects of live load and dead load and dynamic coefficient which are the main criteria. The value is greater than 1 so it is safe to use for the traffic. Then the five different scenarios revealed that the deteriorated condition of the super structure is the main factor which can reduce the value of the bridge safety factor very large (1.05).

Finally, from the overall results it concluded that the bridge is safe to use for the present traffic load but for the future traffic load, change the damaged elements, apply the coating for to resist the deterioration and good aesthetic appearance it is necessary to repair, because this bridge passed more than hundred years.

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