Strengthening and repair of steel bridges

Techniques and management

Master of Science Thesis in the Master's Programme Structural Engineering and Building Performance Design

HASAN DEMIR

Department of Civil and Environmental Engineering
Division of Structural Engineering
Steel and Timber Structures
CHALMERS UNIVERSITY OF TECHNOLOGY

Göteborg, Sweden 2011
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Cover:
Maintenance of a bridge in Blekinge in Sweden.
BaTMan.

Chalmers reproservice
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ABSTRACT

In modern society, well designed and working infrastructure is important. For this reason large investment is made/making to build roads, railways and of course bridges. Bridges play a key factor in road and railway networks. At the present time, road authorities deal with a large stock of old bridges that are in need for repair and strengthening.

Besides aging, there are several reasons that necessitate strengthening and repair operations such as damage caused by accident, vandalism, increasing service load etc. Therefore repair and strengthening works are vital to keep existing bridges in service and also traffic safety.

The focus of this thesis is on strengthening and repair of metallic bridges. Firstly metallic materials and their performance are presented. Different kind of tests to identify existing materials and damage are introduced in continuation. Common type of damage and remedial work which include common strengthening and repair works are explained. To make it easier to understand remedial works and different situations some strengthening and repair cases are studied.

In the last of the thesis, management of bridges in Sweden using Bridge and Tunnel Management Programme (BaTMan) used by Swedish Road Administration is presented in this part. Different issues about new and existing bridges are included in this part.

Key words: Strengthening, repair, remedial works, BaTMan, destructive tests, non-destructive tests, cast iron, wrought iron, steel.

Förstärkning och reparation av stålbroar
Teknik och förvaltning
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SAMMANFATTNING

I det moderna samhället, är en väl utformad och fungerande infrastruktur viktig. Därför har stora investeringar gjorts för att bygga vägar, järnvägar och naturligtvis broar. Broar spelar en väsentlig roll i väg- och järnvägsnäten. För närvarande har myndigheter ansvar för ett stort antal gamla broar som är i behov av reparation och förstärkning.

Förutom åldrande finns det flera orsakar att utföra förstärkning och reparationer på broar, såsom skador som orsakas av olycka, skadegörelse, ökning av belastning etc. Därför är det viktigt att reparera och förstärka broar dels för att hålla befintliga broar i stånd, dels från trafiksäkerhetssynpunkt.

Fokus i denna avhandling är förstärkning och reparation av metalliska broar. Först presenteras metalliska material såsom, stål, gjutjärn och smidesjärn och deras materialegenskaper presenteras. Därefter beskrivs olika typer av tester för att identifiera befintliga material och skador. Vanlig typ av skador och olika typer av förstärknings och reparations arbete introduceras. Ett antal förstärknings- och reparationsarbeten som gjorts i verkligheten har studerats och presenteras för att underlätta förståelsen av olika metoder.

I den sista delen av arbetet presenteras förvaltningen av broar i Sverige med hjälp av BaTMan (Bridge and Tunnel management programme) som används av Trafikverket. I den delen diskuteras olika frågor om befintliga och nya broar.

Nyckelord: Förstärkning, reparation, BaTMan, förstörande test, icke förstörande test, gjutjärn, stål, smidesjärn

Contents

Preface

In this master thesis, three main areas are studied, different remedial works, technique and management of bridges, issues of different strengthening and repair activities respectively. This project is carried out from April 2011 to November 2011 at the Department of Structural Engineering, Division of Steel and Timber Structures, Chalmers University of Technology, Sweden.

First of all, I would like to thank my supervisor Reza Haghani and my examiner Mohammad Al-Emrani who always was available and helped and supported me to finish this thesis. I am also grateful to Valle Janssen and Adriano Maglica for all helps in this thesis.

Last but not least, I would like to express my greatest thanks to my family and friends to support me during my thesis.

Göteborg November 2011

Hasan Demir

1 Introduction

1.1 Background

Bridges play a vital role in road and railway networks all over the world. They are often subjected to variable load patterns, i.e. increase in axel load and traffic intensity by time, and sometimes harsh environments which might result in the loss of their functionality.

According to reports published by road and railway authorities, many bridges in different countries are considered to be either structurally deficient or obsolete.

For instance in USA 122 000 bridges out of 615 000 bridges in the country are in need for upgrading. Similar problem exists in Europe where 66 % of the bridges are 50 years or older [6]. This indicates that many of existing bridges are not compatible with existing traffic in terms of axel loads and traffic intensity which might give rise to problems such as load carrying capacity and fatigue.

Two strategies which might be taken when it comes to structurally deficient bridges include; (1) replacement of a bridge with a new structure and (2) upgrading of the structure to a desired level. The first option is usually very expensive and involves a great deal of traffic disturbance. The second option, which includes strengthening and repair of structurally damaged or weak elements, is on the other hand a more local approach which targets the points causing the total deficiency of the structure. The latter approach is of course, more economic and causes less disturbance in traffic. The decision of using which strategy is however, dependent on the results of cost and life cycle analyses.

As mentioned, a large number of bridges around the world are in need for repair or strengthening. Main reasons for repair of bridges might be aging, accident damage, aggressive environmental factors, etc. Some of the reasons which might necessitate strengthening of bridges are increases in service loads and intensity, changes in codes/standards or extra safety requirements. In Sweden, Swedish Road Administration (TRV) manages/owns around 20176 road and railway bridges (15959 roadway bridges and 4217 railway bridges). 53 % of railway bridges and 35% of roadway bridges in Sweden are older than 50 years [16].

Management of strengthening and repair of a large stock of bridges is usually a very complicated process and involves many engineering and management elements.

A glance at amount of money spent by road and railway owners to keep their bridges in service at standard levels, reveals the importance of the fact that strengthening and repair operations should be optimum in terms of design, efficiency and application. For example, within a period of 10 years (1999 to 2009) TRV spent approximately 152 million SEK for maintenance, road measures and improvement of bridges in Swedish railway network. For roadway bridges, SRA spent around 6 billion SEK to keep bridges in service [16].

Therefore, it is important for structural engineers working in this area, to have a clear picture about damages and remedial work for steel bridges.

1.2 Aim

The aim of this thesis is that give an overview of defects/problems and different remedial work of existing metallic bridges built of cast iron, wrought iron and steel. In this thesis it is also anticipated that describing the different common defects and tests for identifying and detecting defects.

A questionnaire worked together with Reza Haghani is included in this thesis. The questionnaire is filled by bridge experts and additional information about bridge management is taken and brought in the thesis.

1.3 Limitations

The study is limited to the existing metallic bridges of cast iron, wrought iron and steel and also to the literatures from UK and Poland. Described tests, defects, remedial works and case studies are chosen from the studied literatures and they are commonly used in bridge structures.

The questionnaire is limited to three bridge experts from different countries. Some additional information is brought in this report from BaTMan.

1.4 Method

After some discussions and meetings between my supervisor and me, decision is taken that the thesis can start with literature views. During the literature views, interviews are also ranged and different authorities are contacted.

Finally, by using BaTMan and studying different manuals from BaTMan, techniques and management of existing bridges in Sweden are investigated.

1.5 Structure of the report

Chapter 2 describes different materials and their performances. In chapter 3 different tests are presented such as material identification, destructive and non-destructive tests. Different types of bridges from studied literature are presented in Chapter 4.

In chapter 5 common defects are presented.

Chapter 6 explains general principles of remedial works and different remedial works for metallic bridge structures where repair or strengthening of bridges is issue.

In chapter 7, 11 study cases are chosen among 39 studied cases and additional 7 different bridges are studied where the painting and surface damages are an issue, are presented.

In chapter 8, management of Swedish bridges by help of BaTMan is presented. In this chapter, issues of different strengthening and repair activities are also presented.

2 Material

Cast iron, wrought iron and steel are most commonly used materials in metallic bridge structures. Aluminium is also developed and used in bridge structures but there are only a few bridges of aluminium. In this master thesis only cast iron, wrought iron and steel are described and taken into account.

2.1 Cast iron

Cast iron came to use in the beginning of 1700 and the first iron bridge was done in 1800. Its name was Ironbridge in Coalbrookdale, UK, see figure 2.1.

Figure 2.1 The Iron Bridge in Coalbrookdale, UK [17].

There are two different iron types that can be manufactured, white iron and grey iron. When the carbon in cast iron seems like a cementite, the cast iron is called white iron and fracture of this iron has white look. White cast iron is hard and brittle and is used in bridge constructions [1].

Grey cast iron contains two to five per cent carbon and also some silicon. These alloying elements influence properties of grey cast iron. Another factor which is also influencing the material properties is rate of solidification. Most of carbon in grey cast iron presents as flakes of graphite and the rest of carbon presents of pearlite and ferrite. Grey cast iron has grey surface when it is fractured. Grey cast iron is softer than white iron and ductile [1] [4].

Cast iron is strong in compression and weak in tension. It is also brittle and sensitive for fatigue and shows size effect [1] [4]. As it can be seen in Figure 2.2, cast iron has non-linear relationship between stress and strain because of presence of graphite flakes. It can also be noticed that there is no yield point which can be defined accurately [1].

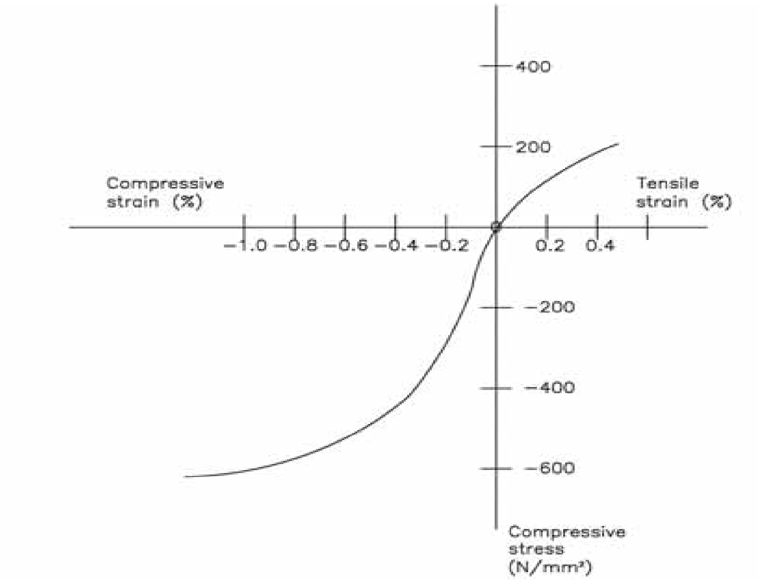


Figure 2.2 Stress-strain relationship for cast iron [1].

Ultimate failure in compression and tension is approximated by maximum shear strain energy (Von Mises) criterion and principal tensile stress (Rankine) criterion respectively. The presence of casting defects and high residual shrinkage stress make cast iron sensitive for fatigue [4]. Cast iron has also good corrosion resistance, see Figure 2.3 [1].

Figure 2.3 Cast iron after 50 years without maintenance, corrosion is evident [1].

2.2 Wrought iron

Process, to convert pig iron into wrought iron is called puddling process. In this process reheated iron is stirred in a furnace to eliminate carbon and other contaminations. After the cooling of the iron, the iron is reworking by reheating, hammering and rolling into bar. The repeating process or reworking gives improved slag stingers, means that better strength and ductility properties of the wrought iron is kept [1] [4].

The performance of the wrought iron is higher than cast iron, better tensile strength and ductility. The wrought iron is more corrosion sensitive than the cast iron [1].

The stringer in the wrought iron is placed in the plane of a plate or the axis of a rod. This gives the material anisotropic properties. In comparison, the strength parallel to the stringers is one third more than the strength perpendicular to the stringers. In Figure 2.4, a typical stress-strain relationship of wrought iron can be seen. The characteristic of the wrought iron is similar in tension and compression [1] [4].

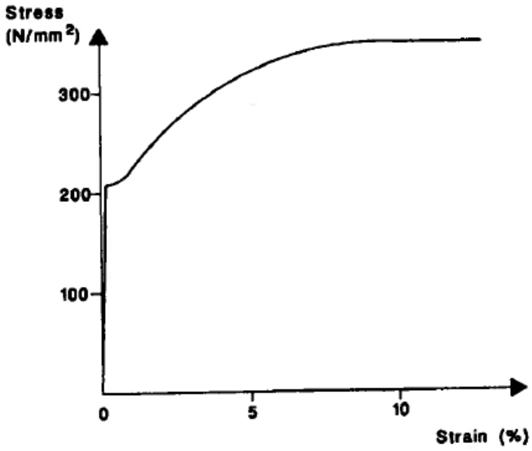


Figure 2.4 Stress-strain relationship for wrought iron [4].

The assessment of the fatigue behaviour of the wrought iron is difficult because of it depends on the cold working in final steps of production. Typically propagation of the cracks starts from rivet holes or from large stress concentrations [4].

About 1000 rail bridges with beams made of wrought iron in 1950s were grouped by former British Railway, had risk of fatigue failure. During reconstruction or replacement, different beams from different bridges were tested in fatigue. The test result indicated that the beams had different production properties and ages compared to each other. In most cases, rivet holes governed to the crack initiation and consequently the fatigue life [1].

2.3 Steel

Mechanical properties of steel comprise heat treatment and production process. Amount of the carbon in steel can be up to 2.5 per cent and more than 2.5 per cent carbon content, the material is classified as cast iron. Adding the carbon to iron raises its strength but lowers its ductility. Structural steel is produced with 0.1 to 0.25 per cent carbon to get an improved combination of strength and ductility, see Figure 2.5 [1].

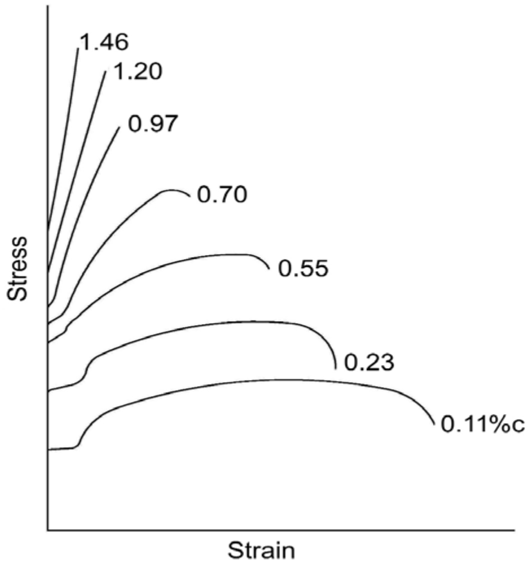


Figure 2.5 The influence of carbon content on behaviour of steel [1].

The quality of the steel up to 1950 was poorer than what is produced today and laminations, deformities and inclusions were common defects for early steel. Due to low fracture toughness and high transition temperatures the early steel bridges were brittle. In March 1938 a bridge at Hasselt in Belgium collapsed 14 months after construction. The fracture initiated in a weld at -20°C. During 1940s all welded liberty ships suffered from brittle fracture and under 1950s research works in improvement of manufacture of steel and welding process were started [1].

The lower carbon content in the modern steel makes the material have a linear-elastic relationship between stress and strain. In Figure 2.6 upper and lower yield points can be seen as well as and the ductile-plastic area [1].

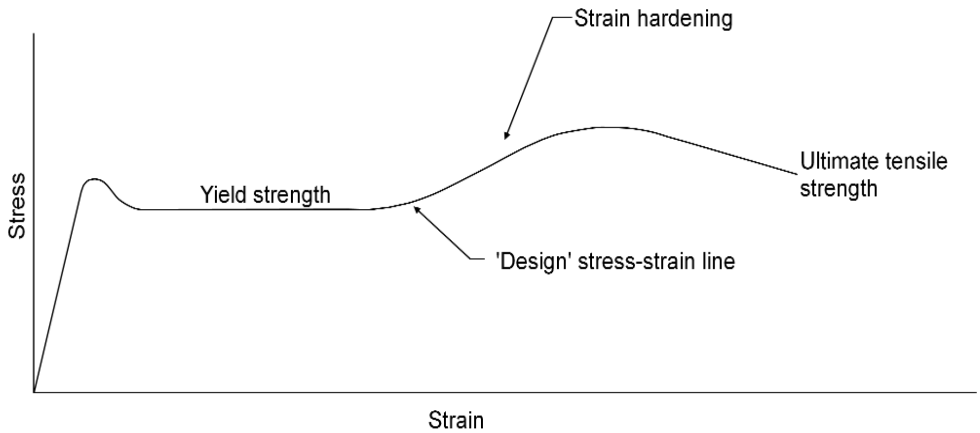


Figure 2.6 Stress and strain relation in modern steel [1].

The steel has more tendencies to corrode than cast iron and therefore steel must be painted repeatedly. In rural environment the rate of the corrosion of steel is about 0.05 mm loss of thickness per year. This corrosion rate is between 0.05 and 0.1 mm in industrial environment [3].

3 Tests

There are a lot of tests for metallic materials to identify, detect and examine damages. In this chapter some test methods are chosen and explained from the studied literatures. The literature is published in UK and therefore these methods are most common in UK.

3.1 Material identification

When a bridge has not sufficient load bearing capacity or damaged and strengthening is needed it is important to identify and understand the material and their performance. Surface etching, the chip test, the spark test and chemical test are explained briefly below.

3.1.1 Surface etching

This method is very straightforward. First of all, the surface is ground and then polished and finally, some part of the section is etched. The examination of the etched material or member is made with use of microscope by an experienced person [1].

Different materials have different microstructure and this property is helping to identify the materials. Figure 3.1 shows microstructure of cast iron and wrought iron.

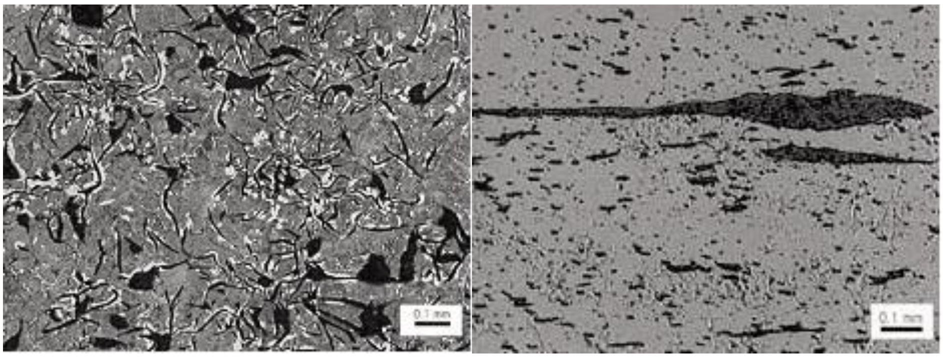


Figure 3.1 Microstructure of cast iron on the left side and wrought iron on the right side [1].

Steel has a granular structure, mostly of pure iron with small volumes or pearlite [1].

3.1.2 The chip test

A small piece of the metal is removed by help of sharp cold-chisel. Colour on the fracture surface and the shape of the edges of removed chip reveal and identify type of material. Table 3.1 shows material properties for cast iron, wrought iron and steel.

Table 3.1 Properties of the metal in the chip test [1].

Metal	Fracture colour	Fracture appearance	Chip characteristics
Cast iron	Dark grey	Crystalline	Small, brittle, rough
Wrought iron	Bright grey	Fibrous, non-crystalline	Long, smooth
Steel	Light grey	Finely crystalline	Long, smooth

3.1.3 The spark test

A piece or surface of the material is ground by a rotating wheel to create a spark test. Then the created spark pattern is investigated by eyes. Chemical properties of the material influence the spark pattern. Cast iron creates an enormously short spark stream with widespread branching. Wrought iron creates no branching but the length of a spark is twice compared to cast iron. Steel has also a long spark. The content of carbon in steel makes spark pattern different [1]. The spark test results for wrought iron and mild steel can be seen in Figure 3.2.

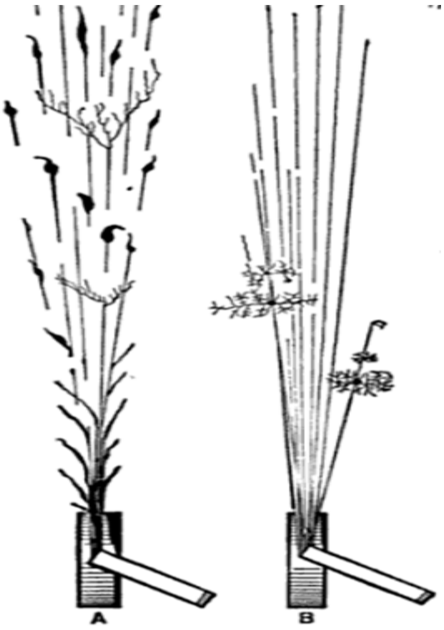


Figure 3.2 Result of the spark test for wrought iron on left side and mild steel on right side [25].

Two disadvantages are involved in spark test. The first is that the test requires a high level of experience to ensure that the spark pattern can be identified correctly and the second is the dependency on tools. The speed and cleanliness of the wheel and finally, the pressure on the test material, influence the result of the spark test [1].

3.1.4 Chemical analysis

It is suitable to use chemical analysis to identify material and its properties for welding, for example if it has been welded or is to be welded. Optical emission spectrometry (OES) is the most common method. To make this method, a piece of material is to be removed or taken from structural component and send to a laboratory. This sample can be taken from the structural component by drillings, chips or slivers of material. The sample should be weighed 15 g or alternatively a flat sample at minimum 12 mm x 12 mm in area and 2 mm thick. It is recommended that take a sample more than one place and choose different places through thickness [1].

There are in-situ instruments for doing this analysis. Instruments based on X-ray fluorescence are cheaper compared to the OES method but these will not give result for carbon and sulphur which are of interest. The OES method can be used in-situ but the result will not be accurate as in laboratory and as mentioned before to use this method in the field is expensive [1].

3.2 Non-destructive tests

3.2.1 Hardness and strength testing

The hardness of materials can be measured by Vickers, Rockwell and Brinell tests. For steel there is a relation between hardness and tensile strength of the materials, so out of hardness test, tensile strength can be estimated. This relation is not similar in case of cast and wrought iron.

In Vickers test, a pyramid indenter is pressed in the test material under a load of 10 kgf and the penetration in the material is measured. This data is expressed in HV/10. The result can be approximated to estimate tensile strength of the material [1]:

$$Fu = 3.15 \times HV/10$$

Then, the ultimate tensile strength is known, the yield strength can be approximated by relationship below:

$$Fy = 0.55 \times Fu$$

In Brinell hardness test the procedure is similar to Vickers hardness test but the difference is that a hardened steel ball indenter used instead of a diamond pyramid indenter [1].

3.2.2 Magnetic particle inspection (MPI)

Magnetic particle inspection is a test that used to detect a surface defect or a defect in the material near the surface. This test is very common used to detect cracks in welds. The material will be inspected, should be made of a ferromagnetic material due to the test is containing magnetization of material locally. The defect or flaw can be observed by the leakage of the flux. It means that if there is a flaw on surface or near surface, which is not parallel to the flux, the flux will leak from the flaw [1].

Firstly magnetic particles let flowed over the surface. Then the flowed particles are magnetised on the material surface and these particles will take a place on the flux leakage. Finally the crack can be detected visually see Figure 3.3 [1].

Usually these particles are suspended in a liquid hydrocarbon in order to increase their fluidity and they might be also coloured to provide high contrast. In case of highest resolution demanded, the fluorescent coated particles are used with an ultraviolet light source [1].

There are several factors that influence the success of this testing technique. Magnetic flux should be sufficient in the tested surface, conditions of lightning and the operator has experiences see Figure 3.3. The surface condition of the tested material is also influencing the success of the test [1].

Figure 3.3 Operation of MPI test [23].

3.2.3 Dye penetrant testing

Dye penetrant testing is a non-destructive test which is used to detect surface breaking flaws. Firstly, the surface of the material (that will be inspected) is cleaned. Then dye penetrant is put on the surface of the material and the penetrant is drawn into the flaw by capillary action. The surface is cleaned again and allowed to dry. Finally, an absorbent developer is applied over the surface to make a stain on the surface and the flaw is examined. This test cannot be used to porous material [1].

To success the test, the operator shall follow the each step of the procedure carefully and the light condition is also important factor [1].

Different steps of the test can be seen in Figure 3.4.

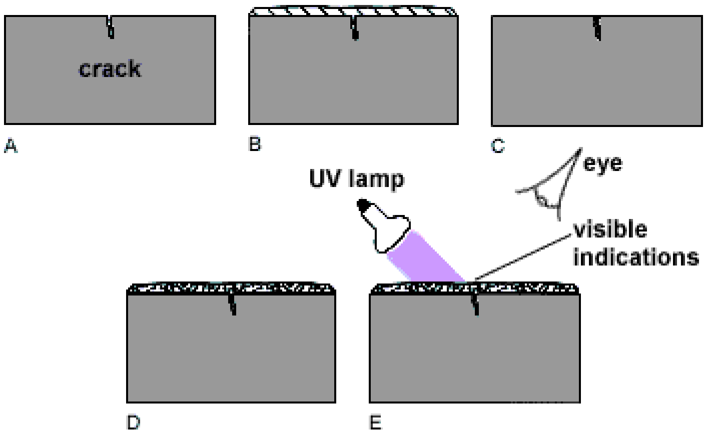


Figure 3.4 Dye penetrant test [24].

Dye penetrant test can be used to identify [1] [7]:

- Grinding cracks
- Heat-affected zone cracks
- Poor weld penetration
- Weld cracks
- Heat treatment cracks
- Fatigue cracks

- Hydrogen cracks
- Inclusions
- Laminations
- Micro-shrinkage
- Gas porosity
- Hot tears
- Cold joints
- Stress corrosion cracks
- Intergranular corrosion

3.2.4 Eddy current testing

Eddy current testing is a testing technique to detect surface and subsurface breaking flaws. A probe is applied on the surface of the material that will be inspected. The probe has an electrical coil that is creating magnetic field. This magnetic field brings a localised electric current in the component and then it creates its own magnetic field. If there is a flaw in the component, the current will be disturbed [1].

Success of the eddy current test depends on how the operator reads and estimates the signal variations [1]. Eddy current test can also be used for [8]:

- Crack detection
- Material thickness
- Coating thickness
- Conductivity

Table 3.2 shows the advantages of the eddy current test [1] [8] and its limitation [8].

Table 3.2 Advantageous [1] [8] and limitations of the eddy test [8].

Advantages	Limitations
Subsurface defects can be found	Inspection can be applied to only conductive materials
Immediate results	Surface accessibility (for probe)
No need to contact between the probe and the surface	Skill and experience is required (more than other techniques)
Portable equipment	Roughness of the surface is influencing the test
Can be used more than flaw detection	Reference standards needed
Required preparation of the component is minimum	Penetration depth is limited

Complex shapes and sizes of conductive material can be checked	Flaws parallel to the probe can be invisible
No dye and magnetic liquid (clean test)	

3.2.5 Ultrasonic testing

Ultrasonic testing is used to detect surface and surface discontinuities. Material thickness can also be measured by this test. By pulser/receiver, short pulses of very high frequency sound energy (1 MHz to 100 MHz) is applied and propagated in the component. If there is a discontinuity (e.g. crack), the signals will be reflected back from the discontinuity surface and then the reflected signal is changed into an electrical signal by a transducer and showed on a screen. The cracks size and place can be estimated out of signal’s travel time related to the distance [1] [8].

Disadvantages of ultrasonic test [1] [8]:

- Cast iron has low sound transmission and high signal noise (due to graphite flakes) and therefore ultrasonic test is not useful
- High requirement of skill and training of the operator
- Accessible surface is needed to transfer ultrasound
- The test is not useful for rough, irregular, very small, very thin or non-homogeneous materials
- Defect parallel to the sound beam may not be identified
- Coupling medium is usually required
- Requirement of reference standards for calibration of equipment and the characterization of cracks

3.3 Destructive tests

3.3.1 Tensile test

Tensile test describes material behaviour under loads and sometimes is needed when the results of hardness test are not accurate. The tensile test is a destructive test and requires a removed specimen from the structure. The out taken specimen should be in direction of principal stress. The accuracy of the test result is depending on the thickness of the specimens due to properties of the material are varying in material thickness. This is concerning mostly cast and wrought iron because of properties are varying in micro scale [1].

Size of the test specimens is varying depending on different codes and recommendations, but the procedure is almost same for all. Two examples of the specimen can be seen in Figure 3.5 [1].

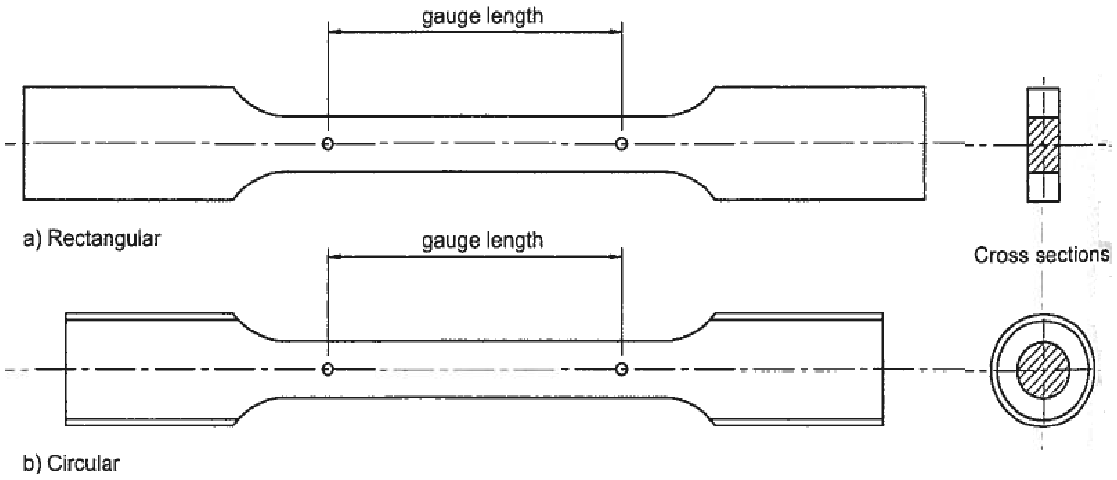


Figure 3.5 Two different examples [1].

Middle section of the specimens has smaller cross-section. The middle cross-section is machined to create a gauge length. The main idea behind the gauge length is to fail specimen in pure tension without any other effects, such as grip effect. In British Standards (BS EN 10002-1), the minimum gauge length is given as 20 mm for standard geometry. The grips and the extensometer are key factors for the geometry of the specimens [1]. An example of extensometer is showed in Figure 3.6.

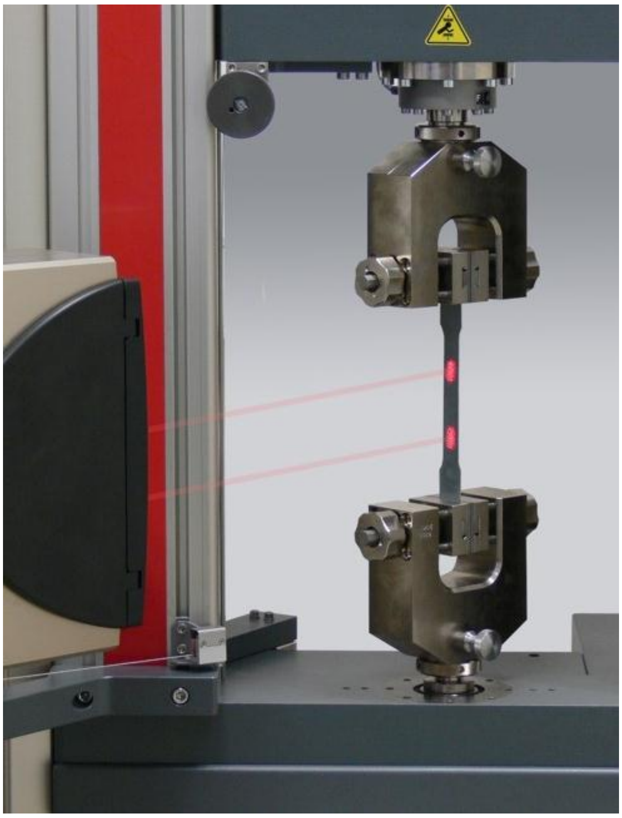


Figure 3.6 Extensometer (non-contact extensometer) [12].

The yield strengths, elongation and tensile strength can be determined by the load-extension curve [1].

3.3.2 Charpy impact test

Toughness of the material can be estimated by Charpy test. Toughness is a material characteristic defined as amount of energy n eeded to cause the material to fracture [9].

There are two Charpy tests, Charpy-V and Charpy-U. The most common is Charpy-V. The specimen that is taken from structural component has the standard dimension of 10 mm x 10 mm x 55 mm with a 2 mm deep V-shaped notch in the middle of the longer dimension (in Charpy-U the notch has U-shape). As it can be seen in Figure 3.7, the specimen is placed in a Charpy testing machine. The direction of the specimen is parallel to the main tensile stress in the specimen. The knife-edged pendulum from known high and known mass is let go and strike the specimen and then the high of the pendulum after broken specimen is measured [1].

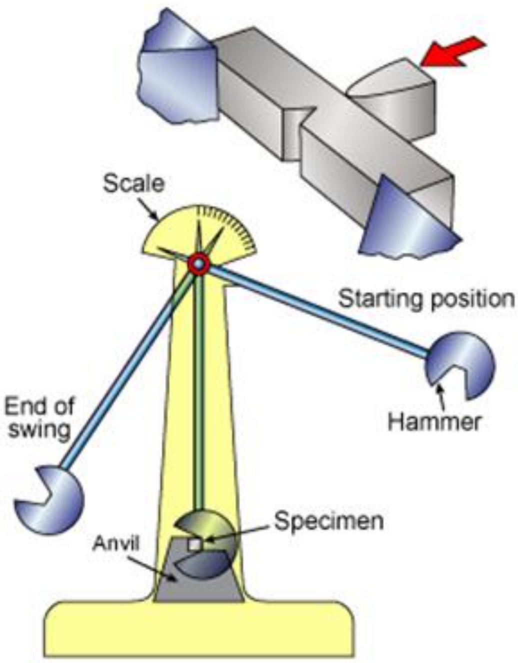


Figure 3.7 Charpy test machine [9].

The high of the pendulum is important because of the higher pendulum causes less energy absorption and give a lower toughness measurement. The temperature is also a vital factor influencing the test. To obtain optimum transition temperature (brittle-ductile transition temperature), more samples and tests are required [1].

3.3.3 Fracture toughness test

This is not a very common test and shall be required due to a presence of the risk of fracture failure. This situation can be come up when there is a fatigue crack in tension region in the structural member. This situation can be caused by fabrication defects or in case of, weld repair is necessary. This test can be used to give a better or more certain data when Charpy test is not sufficient (means containing a marginal data) [1].

3.3.4 Fatigue crack growth test

In the structure, when a fatigue crack is detected by non-destructive test (when the fatigue crack is initiated and not propagated), the fatigue crack growth test is needed to be done. The rate of the crack growth can be determined and facilitate the future maintenance work [1].

3.3.5 Short transverse reduction of area (STRA) test

This test can be required when a new welded component in form of cruciform or T-joint is going to be welded onto old or existing steels. The existing component usually contains contaminations like sulphides and it results in an increase of lamellar tearing in location of or near to heat affected zone. The test specimen that is taken from the existing component is loaded in tension until fracture. The result is examined in percentage due to relationship between original cross-section area and reduced cross-section area [1].

3.3.6 Residual stress measurement

There are different techniques to evaluate stresses in the existing structural component. The stress is comprised of applied stress (dead load and live load) and residual stress from production, for instance weld shrinkage, different cooling rates etc. [1].

The centre hole air-abrasive technique is the least destructive technique among other techniques. In comparison to other techniques, this method is very accurate and trustworthy. The test is standardized and practical applicable. The hole that is drilled on the specimen is very small and is acceptable (not affecting the load path in the material). The hole-drilling test is containing three stages, namely [10];

i. A small hole is drilled in the test specimen, where is in interest

ii. Measurement of the damage around the hole, caused by drilling

iii. Finally, residual stresses are calculated

Another method is current stress measurement (ACSM). This test applies on the out taken sample in laboratory. A special probe is calibrated and used to the out taken sample to measure surface stresses under known applied load. One of the disadvantages of this method is that measurement of the difference between applied dead load and residual stresses. ACSM method is more expansive and requires staff with experience compared to the centre hole air-abrasive method [1].

4 Bridge types

In this chapter different types of bridges and their structural system are briefly introduced. Each type is depending on the length of the span, traffic and width of the bridge. Choice of bridge types is limited to the studied literature.

4.1 Steel girder bridges

Advantage of this type is the simplicity to build. It is possible to cover spans between 5 m and 30 m by this system as shown in Figure 4.1. Spans longer than 15 m are usually not economical [18].

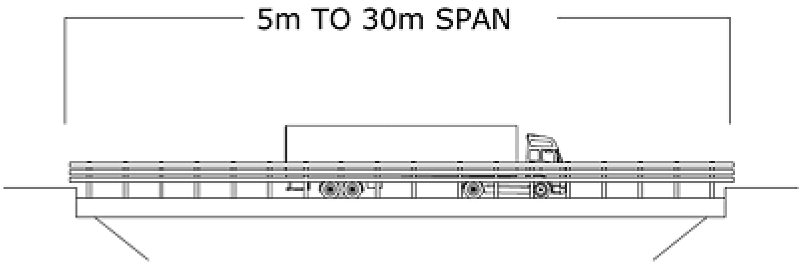


Figure 4.1 Size of the spans for composite steel beam bridges [18].

Another advantage of this bridge type is the great opportunity for selection of the bridge width and also opportunity for widening of the bridge, see Figure 4.2 [18].

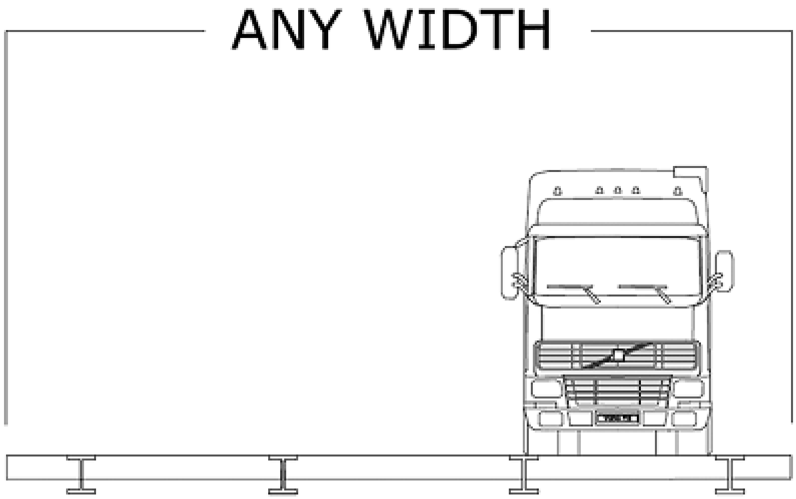


Figure 4.2 Any width can be chosen [18].

The most common used areas are pedestrian bridges and highway bridges. For longer spans, priers are required see Figure 4.3.

Figure 4.3 Erection of continuous steel beam bridge (place and name of the bridge are unknown) [18].

4.2 Over truss bridges

This kind of bridges is very similar to the steel girder bridges, but in this case trusses are used instead of the girders, a typical example can be seen in Figure 4.4 and 4.5. The use of trusses makes the bridge lighter and deeper. The depth of the bridges is approximately 12 % of the span and this should be considered when an over truss bridge is erected over road- or waterway [18].

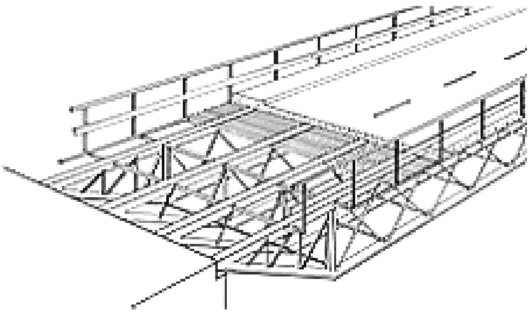


Figure 4.4 A view of over truss bridge [18].

Figure 4.5 Steel over truss bridge in Belize (name of the bridge is unknown) [18].

The span is varying between 20 m and 100 m in this type, see Figure 4.6 and the width of the bridge can be varied depending on wished width. The construction of over truss bridges is more complex and requires more work [18].

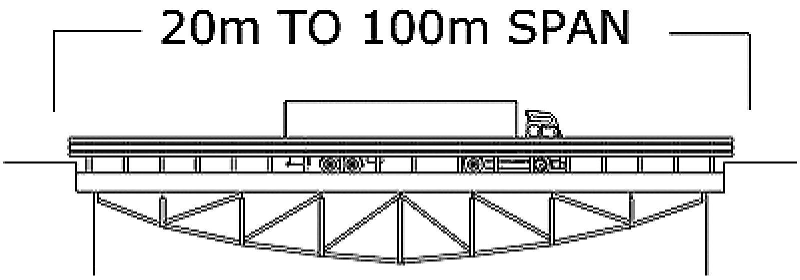


Figure 4.6 A view of the over truss bridge with size of span length [18].

Construction of this bridge can be done in-situ by three different methods. Figure 4.7 shows the construction of the bridge. In this case the bridge is built over causeway or culverts [18].

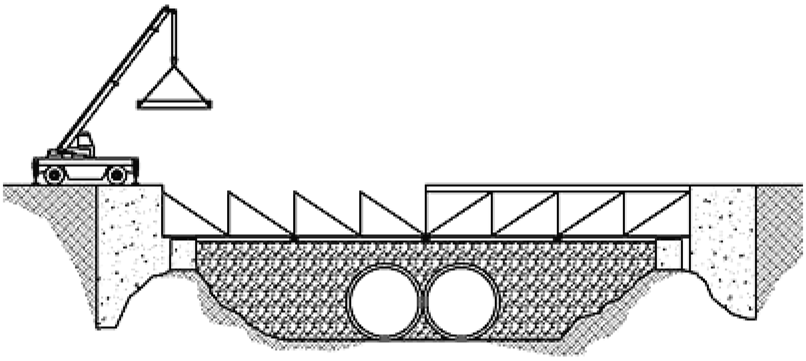


Figure 4.7 Construction of a trough truss bridge [18].

Another method is lifting the pre-constructed beams on the places and joining together at place, see Figure 4.8 [18].

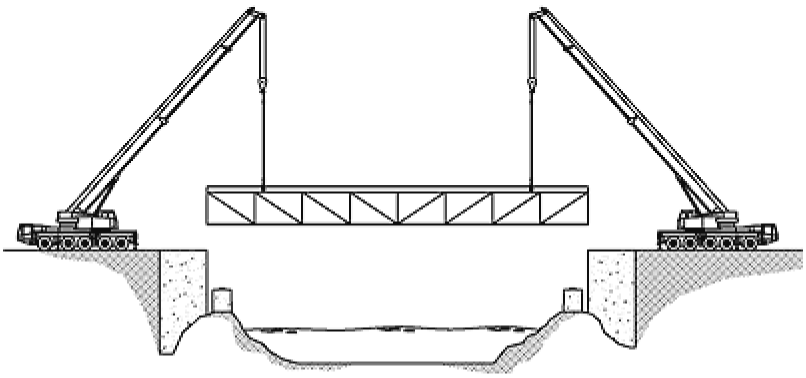


Figure 4.8 Lifting of constructed beams and joining at the place [18].

Roller launch method is also performable in-situ and this method is explained briefly in Chapter 4 and Section 3, see Figure 4.9 [18].

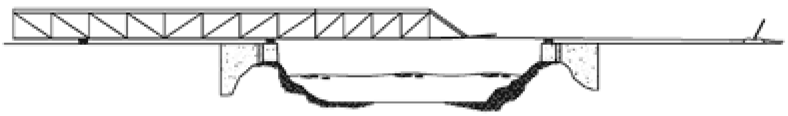


Figure 4.9 Roller launch method [18].

4.3 Trough truss bridges

Trough truss bridges are simple to design and have a modern appearance, see Figure 4.10.

Figure 4.10 Trough truss bridge (name and place of the bridge are unknown) [18].

The span of the trough truss bridges can be up to 120 m and the headroom does not require special level, see Figure 4.11. The width of the bridge is varying depending on wished width [18].

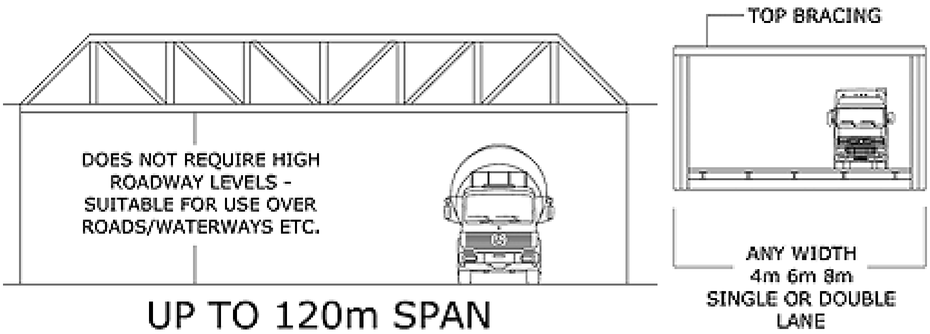


Figure 4.11 A typical trough truss bridge with long span [18].

The light weight and slender structure make this kind of bridges economical. The use of trough truss bridges is beneficial in areas where earthquake is an issue but not recommended in windy areas [18].

The easiest way to build trough truss bridge is in-situ over causeway and culverts and the weather should be dry see Figure 4.12 [18].

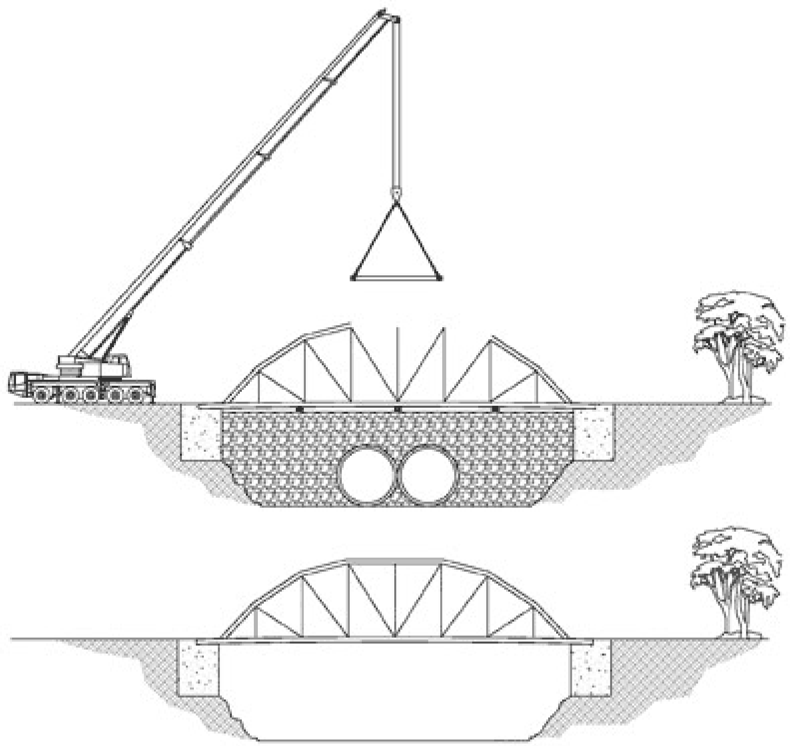


Figure 4.12 Erection of a trough truss bridge [18].

Another method is that a trough truss bridge builds in-situ and then the bridge moves half way by cranes and finally lifts up to the place by cranes see Figure 4.13 [18].

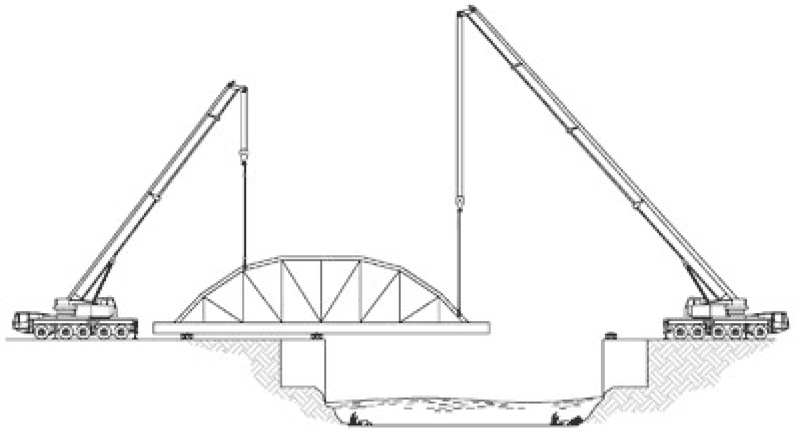


Figure 4.13 Lifting of the trough truss bridge to place [18].

Figure 4.14 shows roller launch method. When the bridge is built in-situ, the bridge is moved by jacks with help of rollers and cantilever technique [18].

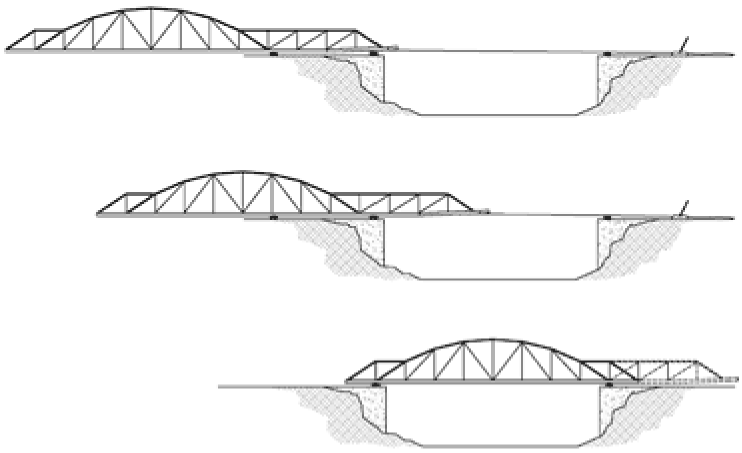


Figure 4.14 Roller launch method [18].

4.4 Steel bowstring Girder Bridge

This bridge type is similar to trough truss bridge but is higher and has curved trusses. The span can be between 50 m and 150 m, see Figure 4.15 [18].

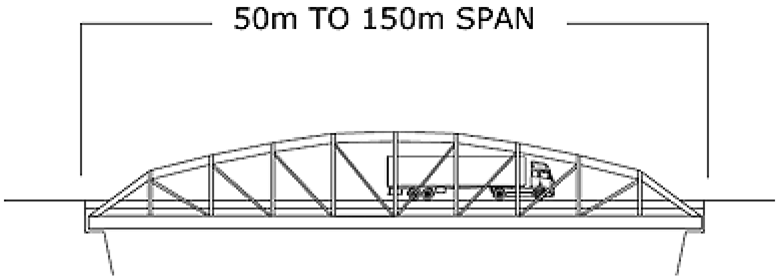


Figure 4.15 A view of steel bowstring girder bridge [18].

This kind of bridges is attractive and can be constructed on uneven soil or less stabile foundation. It can also be pre-fabricated and transported and finally lifted to the place [18].

Figure 4.16 shows a steel bowstring girder bridge that is built in Sudan. Due to lack of big cranes the bridge is constructed by cantilever launch method [18].

Figure 4.16 Steel bowstring girder bridge in Sudan [18].

4.5 Cable-stayed bridge

Cable-stayed bridges are expensive to construct. This kind of bridges is performable when there is no space for prier and no opportunity for cantilever launch. Figure 4.17 shows two different models of cable-stayed bridge. First one is one tower stayed bridge and main span can be from 50 m to 100 m. The second one is two tower stayed bridge and main span can be from 100 m to 200 m, see Figure 4.17. Two tower stayed bridge requires a space of one fifth of main span as outer span [18].

Cable-stayed bridge is light weight and comprising slender structural members. These properties are advantageous in areas with risk of earthquake and disadvantageous in windy areas [18].

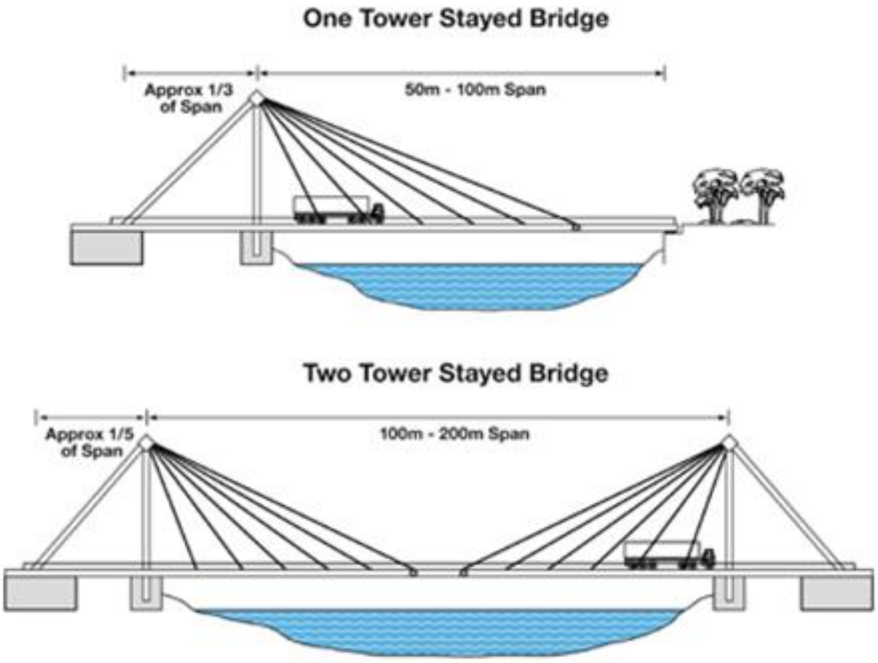


Figure 4.17 Two examples of cable-stayed bridge [18].

Erection of the cable-stayed bridges is illustrated in Figure 4.18, 19, 20 and 21 [18].

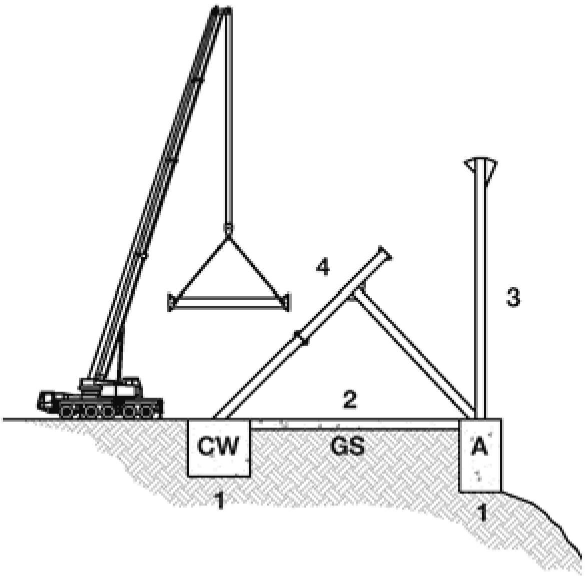


Figure 4.18 Erection of the abutment and the tower [18].

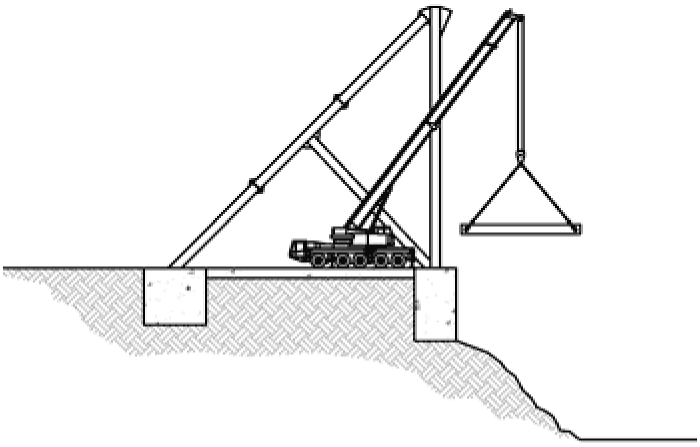


Figure 4.19 Continued erection by deck [18].

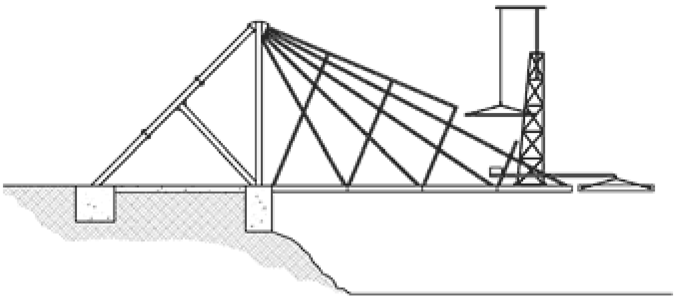


Figure 4.20 Erection cables [18].

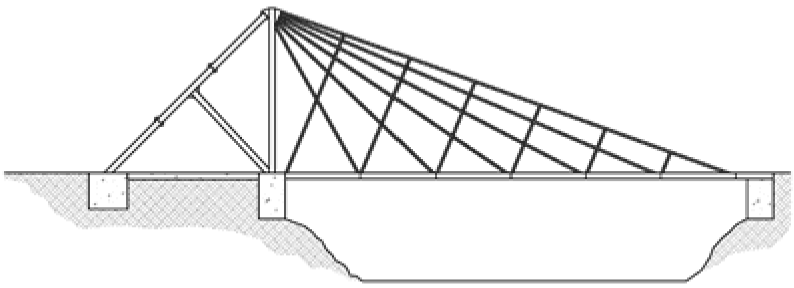


Figure 4.21 A view of completed bridge [18].

4.6 Steel pedestrian bridges

Steel pedestrian bridges are designed for pedestrians and can also be used by cyclists. Span can be between 10 m and 30 m and suitable over waterway, roadway or railway where there is no way for pedestrians and cyclists, see Figure 4.22. Minimum width of the bridge should be 1.2 m due to maintenance.

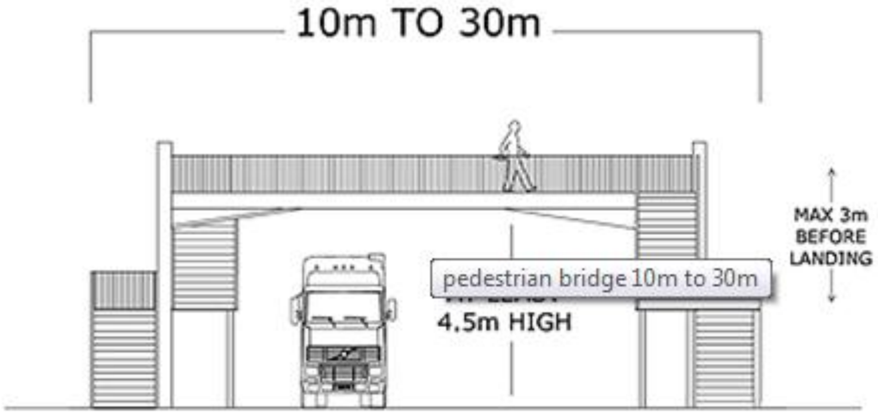


Figure 4.22 A typical pedestrian bridge with minimum headroom and span range [18].

4.7 Composite girder bridges

Composite girder bridges include a bearing system of steel girder and a deck of concrete, see Figure 4.23. The steel structure and the concrete deck are connected to each other and act together in which results in reduction of deflection and increase of strength [18].

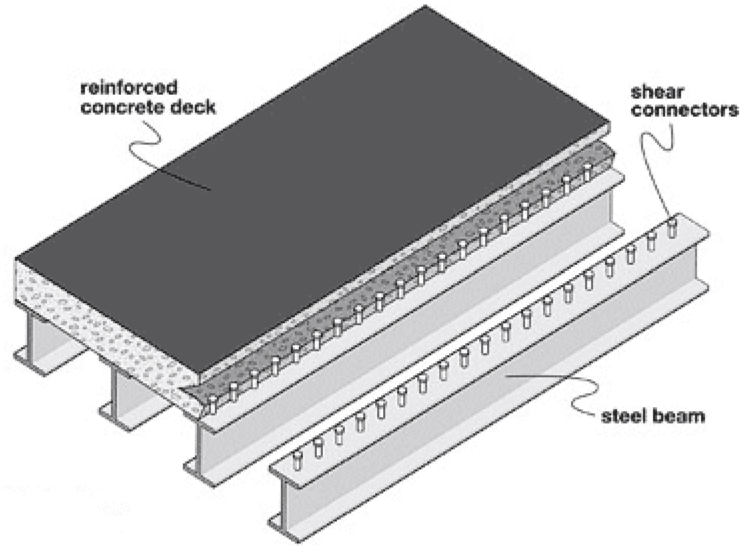


Figure 4.23 Illustration of composite bridge [18].

The connection between the steel structure and the deck can be achieved by a shear connector. Addition of shear connector can be by weld, stud welding machine or nuts and bolts fixed on site see Figure 4.24 [18].

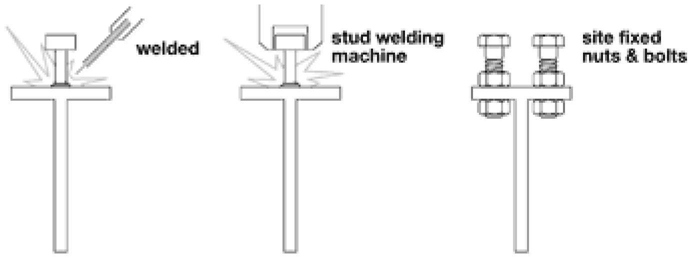


Figure 4.24 Methods of joining shear connector [18].

Composite bridges can be designed in a few ways and two examples are introduced below [18].

In Figure 4.23 simple beam bridges is showed and usually built in span of 8 m, 10 m and 15 m. The simple beam bridge can be built up to 24 m but more than 15 m span is not economical [18].

Figure 4.25 illustrates an over truss bridge with concrete deck. Span can vary approximately from 18 m to 100 m [18].

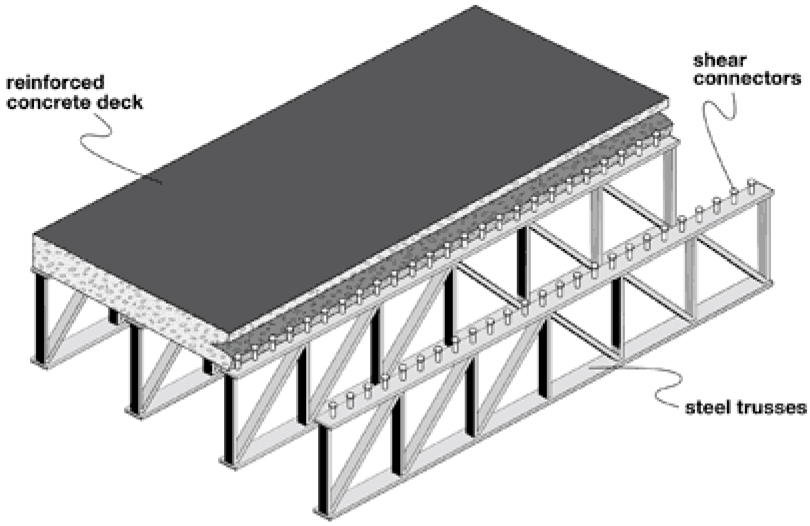


Figure 4.25 A view of over truss bridge with concrete deck [18].

5 Defects

5.1 Material defects

5.1.1 Casting defects

Different flaws can be caused by casting process arise in early cast iron, namely [1] [4]:

- Insufficient venting of the mould can cause blow holes
- Residual stresses (due to various of the cooling rates)
- Cold joints (caused by disruption in casting)
- The material at the centre of the section is weaker and coarser
- Sand contamination from the separated mould
- Section thickness is varying instead of constant thickness
- Deformation and surface flaws
- Cold-spots

5.1.2 Inclusions

The risk for inclusion of slag is an issue when manufacturing wrought iron. These inclusions result in layered and fibrous strands throughout the metal. This micro structure of wrought iron seems like lamella structure can be seen in Figure 3.1. This kind of structure gives the material different properties in different directions. The fibrous inclusion is liable to corrosion and has puff pastry look [1].

In casting process, impurities can occur (e.g., draw holes/gas bubbles, spatter of solidified iron) and these impurities can increase the stresses locally. This can result in crack initiation and can be seen as reason for cracked cast iron. Small defects can cause a total collapse of the bridges. An example for this situation is Inverythan Rail Bridge in United Kingdom. A draw hole in a component of the main beams was not visible in a bolted connection. The draw hole could not be detected and was the reason behind the collapse of Inverythan Rail Bridge, 25 years after construction [1].

5.2 Fabrication defects

Incorrectly repaired castings

Blow holes are most common defect in cast iron and can be repaired in the factory. The reparation can be done by infilling the blow holes with non-structural material. This defect is not easy to detect on-site due to coating and painting [1]. Figure 5.1 shows an example where a defect in a beam that was over 100 years old and could not be detected till 2006.

Figure 5.1 A detect is older than 100 years [1].

Surface defects

Use of the chipped rolls can be a reason to surface defects. Another reason might be manufacturer's name which is rolled into the component. This acts as stress raiser and can be initiation point for fatigue [1].

Rolling

Millscale could occur during the rolling process of wrought iron and early steelwork. Location of the millscale is more common on the surface than embedded. One of the results of the millscale is discontinuity of the material and in turn reduced corrosion resistance. Finally pitting corrosion can occur where the millscale is [1].

When a cold rolling process is done below the softening temperature, strain hardening and reduction of ductility can occur which are not easily detectable in existing structures. This can be a reason behind the other defects such as cracks [1].

Pre-cambering

In steel beams, deflection caused by self-weight and dead loads can be repaired by pre-cambering (by cold-bending). Pre-cambering is not classified as defect but can reduce ductility and cause strain hardening [1].

Misalignment of rivet holes

This issue has no structural consequence but when strengthening concerns, it is difficult to remove the rivet when strengthening is required [1].

Welding

Welding causes a residual stresses in the structure. The residual stresses will affect the buckling behaviour and fatigue life of members.

In welding process, heating and cooling in the steel structure can cause reduction of ductility and defects. Example to defects is sharp notches in the toe or root of the weld [1].

5.3 Corrosion

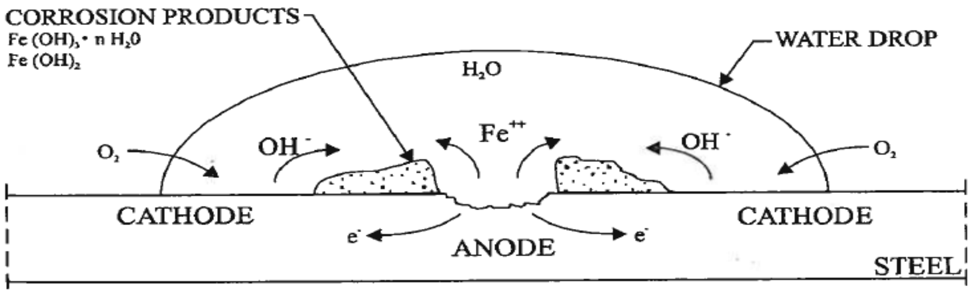
Corrosion is a common problem for steel and iron structures. The product of corrosion has greater volume than the parent material and causes pushing stresses in the material.

Corrosion causes a reduction of the cross-section of the structural element and in turn leads to reduction of stiffness of the structure. Decrease in the cross-section area also means increase in stresses in the structure.

In steel structures, five different types of corrosion could be identified, namely:

- Surface corrosion
- Pitting corrosion
- Crevice corrosion
- Galvanic corrosion
- Stress corrosion

Surface corrosion, occurs on the surface of the component and causes damage and reduction of cross section in the structural element. The process of the surface corrosion is shown in Figure 5.2 [2].



Mechanisms of surface corrosion of structural steel. Basic processes — anode: $2Fe \rightarrow 2Fe^{++} + 4e^{=}$, cathode: $O_2 + 2H_2O + 4e^- \rightarrow 4(OH)^-$. Examples of corrosion products — in case of limited amount of oxygen: $Fe^{++} + 2(OH)^- \rightarrow Fe(OH)_2$, — in case of more free access of oxygen: $2Fe^{++} + 4(OH)^- + 1/2O_2 + (n+1)H_2O \rightarrow 2Fe(OH)_3 \times nH_2O$, $4Fe^{++} + 3O_2 \rightarrow 2Fe_2O_3$.

Figure 5.2 Process of surface corrosion [2]

Pitting corrosion takes place over very small surface and is very difficult to be seen in many cases. Pitting is evolving extremely inside the steel and results in local stress concentration. Humidity has an effect as accelerator for the pitting corrosion [1] [2].

Crevice corrosion occurs when two same type of steels or metals are in contact with each other, for example in bolted reinforcement plates, splice plates, gusset plates, etc. The corrosion product has a swelling effect on material and is leading to tear forces causing damage in the material, see Figure 5.3.

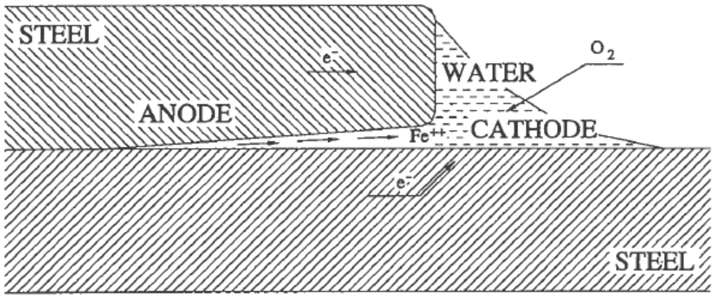


Figure 5.3 Process of crevice corrosion [2].

Galvanic corrosion occurs when two different types of steels or metals are joined or connected to each other, for instance in welded, screwed, bolted or riveted joints. This corrosion causes local damage [2].

Some elements should be present in order for galvanic corrosion to occur, [1]:

- An electrolyte (e.g. water is most common)
- Electrical connection (between the two elements)
- A significant galvanic current (can be enabled by a difference in potential between the metals, at least 50 mV is required [2])
- Cathodic reaction

There are some factors which have accelerating effect on the process of galvanic corrosion. Among them are ambient temperature, the electrolyte's conductivity and the area ratio of the anode and cathode [1]. To prevent the galvanic corrosion, insulation can be put between the metals. This can be provided by coating one metal or both metals, e.g. painting [1].

Stress corrosion is mostly common and concerns cable stayed bridges and cables in suspension bridges [1].

Some details prone to stress corrosion are given below [1]:

- Bearings and expansion joints
- Bearing pins
- Riveted connections
- Connections by cast or built-in
- Tie-rods

5.4 Accident damage

Accident damage caused by vehicles can result in distortion, tearing or cracking in bridge components. Sometime damage on the bridge components do not need to be repaired and left unrepaired depending on how this damage influences the system of load carrying capacity. The examination of the damage should be done very accurately [1]. Some examples to accident damages can be seen in Figure 6.11, 6.17, 7.24 and 7.26.

5.5 Fatigue

Fatigue is a damage that is progressive and localized in structural element. This damage, fatigue, can be occurred when the structural element is subjected to cyclic loading. Failure of the structural element can be occurred under the ultimate load and even under the elastic limit of the material [1].

Fatigue crack has two stages, namely [1]:

- Crack initiation
- Crack propagation

Older structures require special consideration with regard to fatigue since often it was not taken into account in design. Avoiding or reducing the effect of fatigue can be done, for example, by reducing the amount of load and loading cycles, increasing cross sectional area of the member, reducing the load on the member by redistributing the load by adding additional members.

Some details prone to fatigue include [1]:

- Rivet and bolt holes, welds
- Short hangers, in suspension footbridges and road bridges
- Lightweight bracing under vibration loads, wind loads or traffic loads

Some of the details, sensitive to fatigue, can be seen in Figure 5.4 [2].

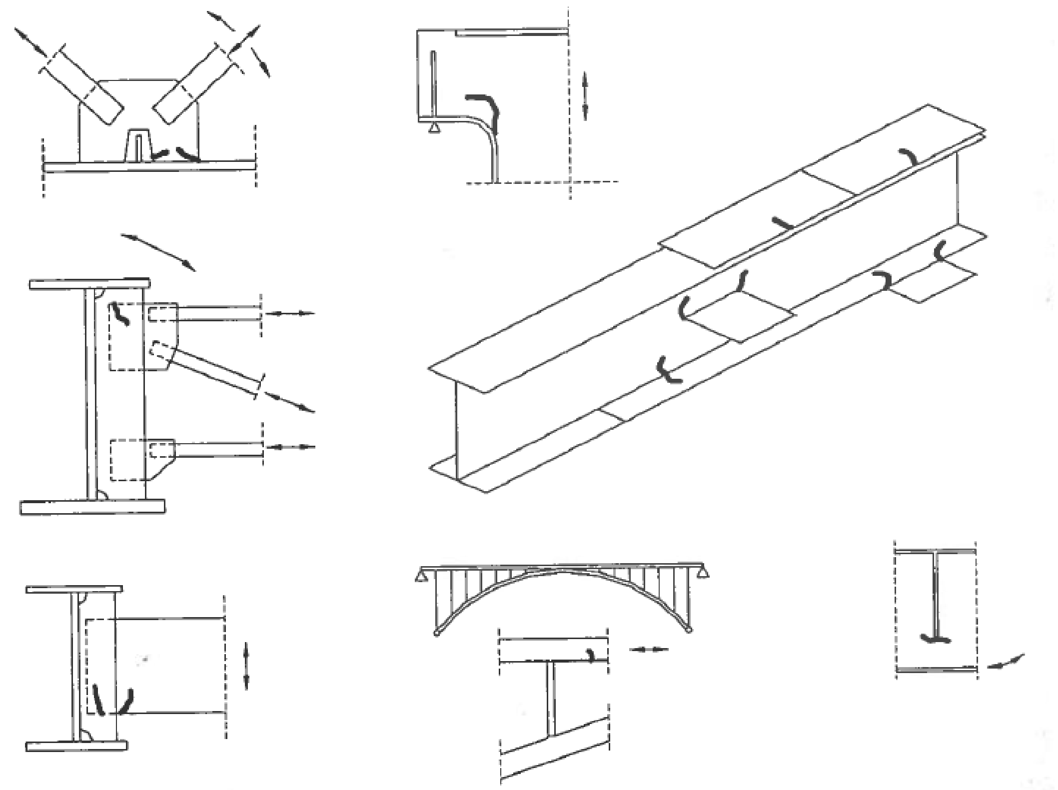
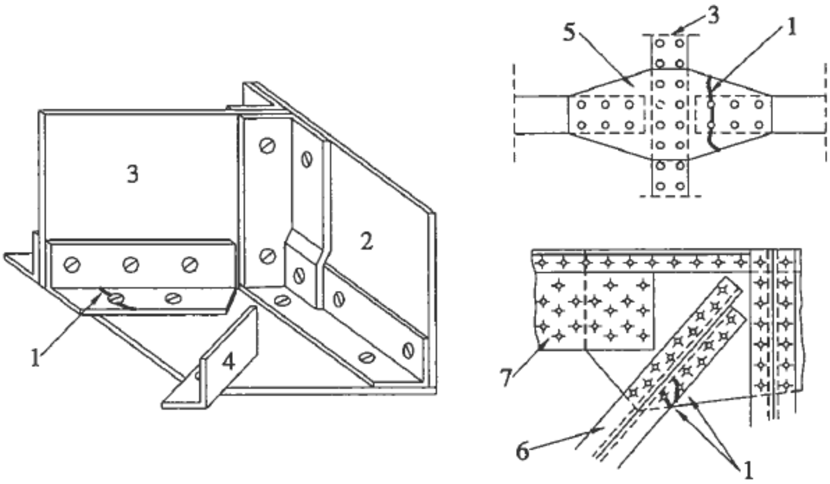


Figure 5.4 Different locations of the fatigue in steel bridge superstructure [2].

In riveted steel truss bridges, the possible location of the fatigue cracks are shown in Figure 5.5 [2].



Possible location of fatigue cracks in riveted truss bridge. 1 — fatigue cracks, 2 — main girder, 3 — transversal beam, 4 — bracing, 5 — cover plate, 6 — diagonal, 7 — upper chord.

Figure 5.5 Fatigue cracks in riveted truss bridges [2].

5.6 External causes of damage

Environment, timber members or decking, wildlife, fire and vandalism are typical examples of external damages. Aggressive environment increases the risk of the corrosion for iron and steel. For instance marine environment is classified as aggressive. De-icing salts are also one of the reasons to corrosion [1].

The acetic acid from deck or member in structures made of timber causes damage of the protection coatings consequently increases the risk of corrosion .In case of wildlife, e.g. the pigeons, the corrosion starts on the bottom flange of steel girders due to guano deposited on the flange [2].

Fire can be caused by vandalism or vehicle incident and results in weakening of the material. Degree of the deterioration should be analysed and examined and repair measures should be applied if required [2].

6 Remedial works

6.1 General principles

6.1.1 Strategies for repair or strengthening

Remedial work can be divided into three categories [1]:

- Reducing the load effects
- Increasing the resistance (not included in this thesis)
- Replacing the component or structure

Repair and strengthening operations can be a mixture of these categories [1]. Repair operations are explained in more detail in section 5 and strengthening operations in section 6 of this Chapter. Reducing the load effects and replacement of the component or structure are briefly explained here.

Reduction of the load effects:

In this case the applied load is reduced or redistributed away from the sensitive component by adding stiffness to different sections [1].

Redistribution of loads should be based on computer analysis or in-situ load testing. The aim of the tests and analyses is to identify the load path in the structure. [1].

The weight limitation of vehicles is also another alternative to reduce the load effects. If additional stiffness to the critical sections results in unacceptable costs, the weight of vehicles can be limited instead. [1].

Replacement of the component or structure:

When the extent of deterioration in a structural member is so that its reparation is not economical and technical, the replacement of the member or structure is considered. Investigation of the time and cost should be done accurately [1].

6.1.2 Heritage issues

There are some considerations under remedial work of heritage structures, namely [1]:

- Minimal use of new material
- Minimal change of look of the structure
- Suitable material and application should be chosen
- Maintenance should be done continuously and not to cause unacceptable damage on the structure
- Any modification does not change the character of the historic structure
- Chosen modification should be reversible
- Archaeological aspect (investigation might be required before remedial work)
- Documentation of the work

6.1.3 Factors in the execution of remedial work [1]

- Safety factors e.g. site workers and public
- Accessibility

- Material availability and delivery
- Size and weight of the material (care to no accident of the delivered material in situ)
- Time plan of the work
- Environmental issues

6.1.4 Public relations

Public interface:

Continuous meeting between contractor and residents should be planed and done to share information about the work and time schedule. For instance noisy work can be informed before. The practical way to inform resident is shared letters [1].

Noise considerations:

The noise from the erection should be considered before the remedial work starting. For example barriers can be installed to provide acoustic protection and more quiet tools can be chosen [1].

Traffic:

Closure of the traffic can be operated under the night for reducing disturbance to local businesses and residents.

6.2 Corrosion removal and surface cleaning

The preparation work of the material surfaces can be divided into several categories, depending on national and international standards or codes. The grades of the surface preparation given below are in use in some countries [2].

> Grade I—all impurities, rust and mill scale are removed. The surface of the cleaned component shall be of metal, uniform and have silvery-greyish look.
>
> Grade II—all impurities, rust and mill scale are removed. A grey oxide layer is tolerable to stay between metal substrate and a mill scale. The allowable limitation of the mill scale is 5% of the area of the cleaned steel surface. The appearance of the cleaned surface is matt and grey.
>
> Grade III—all impurities, the allowable limitation of the mill scale is 20% of the area of the cleaned steel surface. The surface of the cleaned component is non-uniform, and some parts are metallic and other parts in different colours.

Surface preparation of material in grade I can be accomplished by sand or shot blasting and etching, Grade II by sand or shot blasting, hammering, brushing or flame cleaning (also called thermal method) and Grade III by hammering, brushing or scrapping [2].

The hand cleaning process is done by brushing, hammering, scrapping and grinding. It is low efficient and applicable to the small surfaces. Other methods (e.g. blasting) can be applied to larger surfaces and is more efficient [2].

The most usual blasting cleaning methods are namely [2]:

- Dry abrasive blast cleaning
- Wet abrasive air blast cleaning

- Water blast cleaning with or without abrasive

The typical examples for abrasives are sand, grit, cast iron or cast steel shot and wire cut. These abrasive materials can be found in many different grades and sizes, depending on applications. To get an idea about the speed and time of the blast cleaning, an example is given in following. Sand blasting machine with properties of a nozzle diameter of 8 mm and discharge of sand at 13.6 kg/min can clean a surface of 7-13 m^2 per hour if the surface grade is I and 25-37 m^2 per hour if grade II [2].

The blast surface cleaning process is containing hazardous material due to lead oxides from older paint system. Therefore, the vacuum system and enclosure are required and blast particles should be kept away from cleaned surfaces [2]. The chemical and flame cleaning methods are less used in comparison to blast and hand cleaning methods [2]. The organic solvent is applied to the older paint or coating and then the older coating can be removed by hand (e.g. knife). Another example to the remove of the older paint is using special emulsion products. For light rust, numerous inorganic acids can be used (e.g. mostly phosphoric acid). Stronger rust, etching by using acids can be used to remove the rusts [2].

The oxy-acetylene blowpipes are used in flame cleaning operations. This process includes heating of corroded part and removing the softened corroded part. The heating temperature is between 300-500°C. Blowpipe velocity is 1-3 m/min and the flame angle to surface is between 30° and 40°. With flame cleaning, the surface of 6-18 m^2 area can be cleaned per hour. The flame cleaning can cause damage in the structural component to be cleaned. Therefore this process is not recommended to use for thin plates. In steel structures, main element usually is not less than 8 mm and this cleaning method is applicable to remove paint layer [2].

Finally it should be emphasized that corrosion removal and surface cleaning are influencing the behaviour of the anticorrosion protection. Therefore the technical requirements should be followed and met carefully [2].

6.3 Anticorrosion protection

Anticorrosion protection is a very wide topic and depends on national and international standards, recommendations or codes. In general there are two different cases irrespective of the local conditions, namely [2]:

a) Requirement for periodic renewal of the anticorrosion protection without repair

b) Anticorrosion renewal with a large repair

The evaluation of the deterioration is described in instructions. The finish coating is usually easily cleaned without making any damage on the coating and then the view of the coating is compared with the photographs in the standards. Different parts of the coating in different locations in the structure shall be examined [2].

In some countries, there are three different degrees of the deteriorations, namely:

1. Degree I—on the finish coating, loss of gloss, chalking and changes in colour can be detected

2. Degree II—same as degree I and more intensive. Blistering, scaling and cracks in the coating can be detected. Single corrosion point or corrosion centres can occurred

3. Degree III—same as degree II and more intensive.

Anticorrosion protection systems are categorized to; inhibitive primers, sacrificial or galvanic primers and barrier primers [2].

Inhibitive primers prevent the corrosion chemically and mechanically and create a protection in steel against the effect of moisture and oxygen. The primers are applied to the cleaned surface directly. Sacrificial or galvanic primers protect the steel electrochemically by making the surface electrochemically negative means anodic. In this case steel becomes cathodic and the risk of the deterioration is removed. Zinc is the common used material to create this primer. Barrier primers are keeping the steel surface from water, oxygen and ionic material action. The system is containing a few layers of the same materials, as one of the followings below [2]:

- Coal-tar enamels
- Low-build vinyl lacquers
- Epoxy and aliphatic urethanes
- Coal-tar epoxies

Based on the material, anticorrosion protection can be categorized into two main groups, non-metallic coatings (organic coatings) and metallic coatings (inorganic coatings).

Some materials, used as non-metallic coatings are paints, lacquers, asphalt or tar and also epoxy or polyurethane resins. Lacquers and paints are the most common used material and include vehicle, pigment and volatile solvent. Epoxies or vinyls are examples for vehicle and giving the primer cohesion property. Colour of the coating is coming from the pigment. The pigment also is liable for the covering property. The properties of the coatings for instance consistency, durability and strength are influenced by the pigment. The solvent is influencing the bonding of the coating to the steel. Some paints as known oil paints shall not require the solvent .Anticorrosion protection by epoxy or polyurethane resist strongly to chemical and mechanical effects. In some cases hematite can be added to anticorrosion system to give better corrosion resistance [2].

Application of the non-metallic corrosion should be made by experienced staff [2]

Regardless which material used, the ambient temperature should be in the interval +5°C and +40°C and relative humidity less than 90%. The optimum ambient temperature is between +15°C and +25°C. Weather condition is an important factor influencing the process of the painting. Painting is not allowed in rainy and windy weather. First layer is applied to the steel surface is ground layer and has maximum thickness of 50 μm. Application of the ground layer should be made not more than 6 hours after the surface cleaning. The number and thickness of the layers can be found in national or international standards or recommendations [2].

The application of the non-metallic corrosion can be done by three different methods, hand painting with brushes, spray painting and airless spray painting.

Hand painting is an old and traditional method. This method is very simple and does not require energy. Bonding of the paint to the steel surfaces is about 95% and is very high in comparison to other methods. Disadvantage of this method is that it is not very efficient. About 10 m^2 area can be painted per hour. The thickness of a single paint layer in this method is between 20 and 40 μm [2].

Spray painting comprises special equipment; paint-gun, compressor and accessories, e.g., nozzles, hoses, etc. Distance between the steel surface and gun nuzzle, air pressure should be experimentally determined before the painting operation starts and any changing allowed during the painting operation. For instance the air pressure is between 2 and 5 MPa and the distance is between 18 and 25 cm. The bonding of the paint to the surface is less than that of the hand painting, for instance it is about 65% for large steel areas and 20% for truss components. Compared to the hand painting this method is more efficient, 60 to 70 m^2 can be painted per hour. The thickness of a single paint layer is between 10 and 15 μm [2].

Airless spray painting is more advanced compared to the other methods. No air is used. A paint gun with nuzzle (in small diameter) pumps the paint with fine pigment to the surface with pressure of 20 MPa. The painted area is from 200 to 300 m^2 per hour for large steel surfaces. The penetration of the paint particles is very good and gives a good quality almost the same as hand painting. This technique is more economical and provides better quality than spray painting .Anticorrosion protection made of metallic coatings can be carried out by two different methods, namely [2]:

a) Cathodic coatings (means that the used metal has higher electrochemical potential than steel), for examples, copper (Cu), nickel (Ni), lead (Pb) chromium (Cr)

b) Anodic or sacrificial coatings (means that used metal has lower electrochemical potential than steel), for examples, zinc (Zn), cadmium (Cd), aluminium (Al)

In steel bridges anodic coatings are used more than cathodic coatings. There are different application methods of the metallic coatings, namely:

- Electrochemical
- Chemical
- Diffusion
- Metal-vapour plating
- Dipping
- Metallization

The mentioned methods above are usually applied to the new structural elements that are manufactured in factory. In rehabilitation phases the new manufactured elements are going to replace the old elements. Metallization process is most commonly and useful method for large surfaces and the other methods are for small surfaces [2].

Metallization process requires special equipment as spray painting. The equipment is containing a metal spray gun and compressor. The classification of the guns is made of the type of heat sources for melting metals and the following [2]:

- Flame spray guns (gas guns)
- Electric arc guns
- Plasma-arc guns

The flame spray guns and electric arc guns are commonly used in bridge engineering. The thickness or depth of the coating is between 120 and 250 μm depending on environment conditions and durability (e.g., 10, 20 or 50 years) [2].

In general metallization can be divided into three stages without consideration to used material. In the first stage the steel surface is cleaned. The most used method is blast cleaning. When the surface cleaning is done metallization process can be started. The last stage is cooling of the coating. All of abovementioned methods should be done according to recommendations and under control by experienced staff [2].

6.4 Replacement of structural members

A structural member, whole bridge or bearings can be replaced due to damage as long as repair process is not economical and not technically feasible. The whole bridge replacement is not included in this thesis [2].

The main idea behind repair process is that a new structural member is manufactured in the factory and then is installed or assembled in the existing bridge. In this process galvanic corrosion between the new and old structural member can occur and therefore this problem should be considered and avoided. Another issue is jointing, for instance if welding is chosen the weldability of the existing structural member should be investigated. Welds and high strength friction grip bolts are mostly used to join the old and new structural member [2].

Cutting of the damaged member from the existing bridge causes redistribution of the internal loads and consequently the geometry of members in the bridge can change, therefore this should be investigated and analysed. In such cases, if necessary, hydraulic jacks, power winches etc. can be used to support the existing bridge temporarily [2].

From economic point of view the replacement of a deteriorated member is more beneficial than its repair since replacement requires less labour. Time is also a decisive factor in case of bridges with heavy traffic [2].

6.5 Repair of structural members

6.5.1 Stitching

Cracked or fractured metals can be repaired by stitching method. Stitching is applicable to cracked and broken cast iron, aluminium and steel members [1].

The stitching process contains a few of drilled holes on the material. These holes are in line across the crack and also perpendicular the crack. By jig-drilling, holes are opened on the material. Then shaped keys made of high-nickel steel put into the holes. The shaped keys have the same expansion coefficient as the host material (usually cast iron). The reason for choosing high-nickel steel is the shear capacity and the ductility properties. When the stitching across the crack is done, threading process starts for stud screws. These stud screws overlap on each other and fill the crack. Attention should be taken to watertight the holes. Each step can be seen in Figure 6.1 [1].

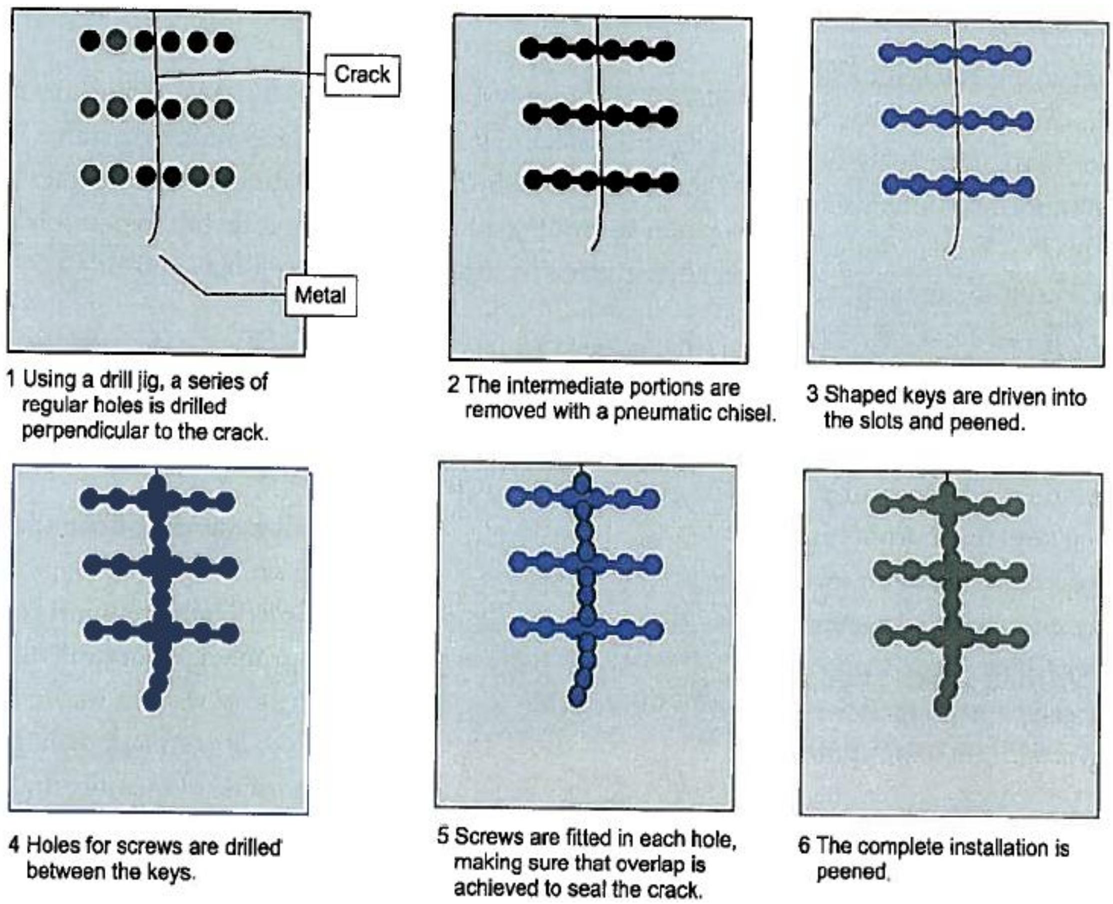


Figure 6.1 Process of stitching [1].

The use of stitching method to repair cracks has some benefits. For example no thermal stresses and distortions are occurred in this repair process [1].

6.5.2 Repair of corroded components

When corrosion occurs and is localised, it is possible to repair this corroded material by removing the corrosion and keep the original and sound material or replace the corroded material. When a replacing method is taken, attention should be paid to load redistributions in the structure under operation. Removed member can cause additional stress to the neighbouring elements and this should be considered in analyses. In Figure 6.2, it can be seen how the localised corrosion is repaired. In this repair [1];

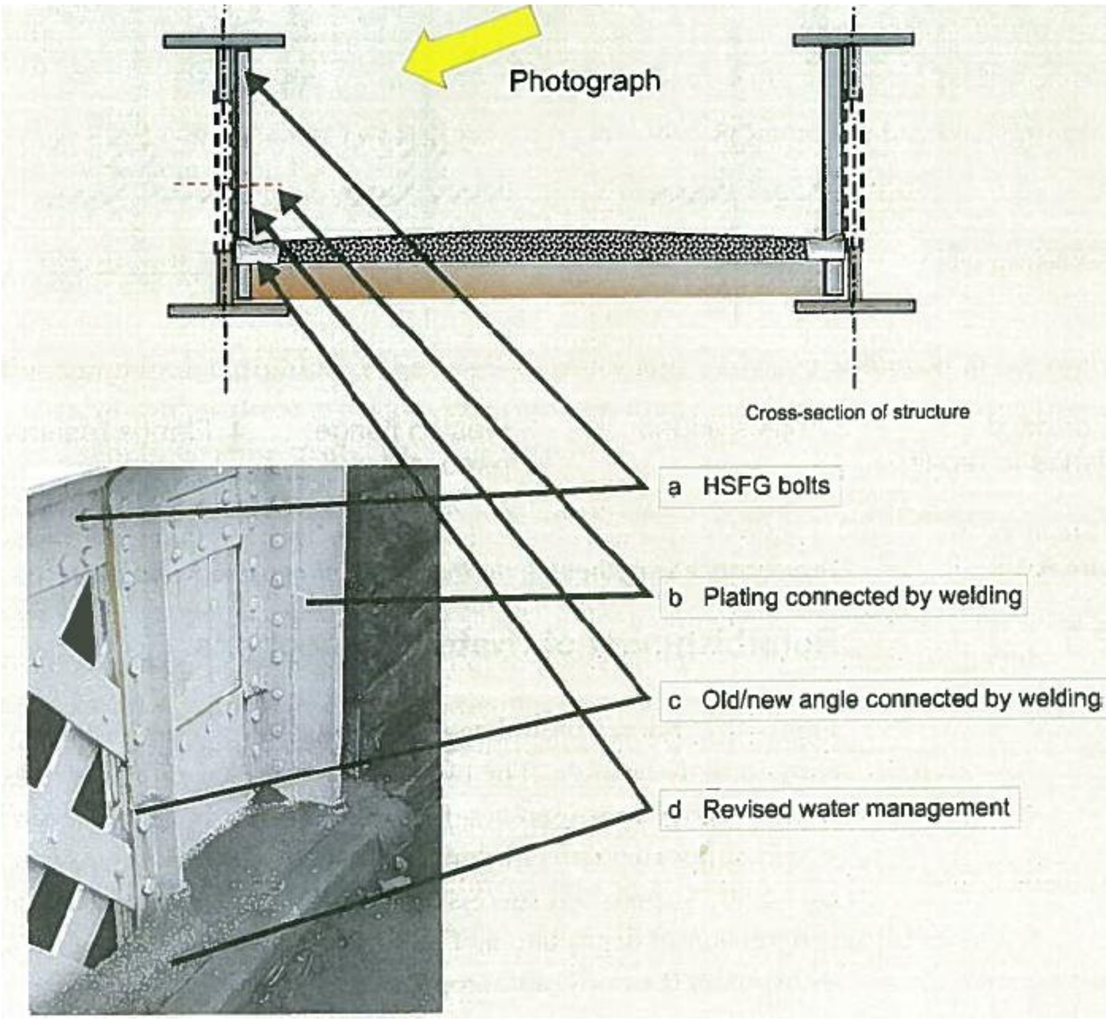


Figure 6.2 Reparation of the badly corroded section [1].

- Original rivets in the connection between the stiffener and web are replaced by HSFG (high strength friction grip) bolts
- Weld and bolt are used to join the upper original plate and lower new plate
- Weld used to join the original and new angle area
- Water management by U-channel

During the repair process, strengthening of some sections might be required when corroded flange is replaced. This example can be seen Figure 6.3 [1].

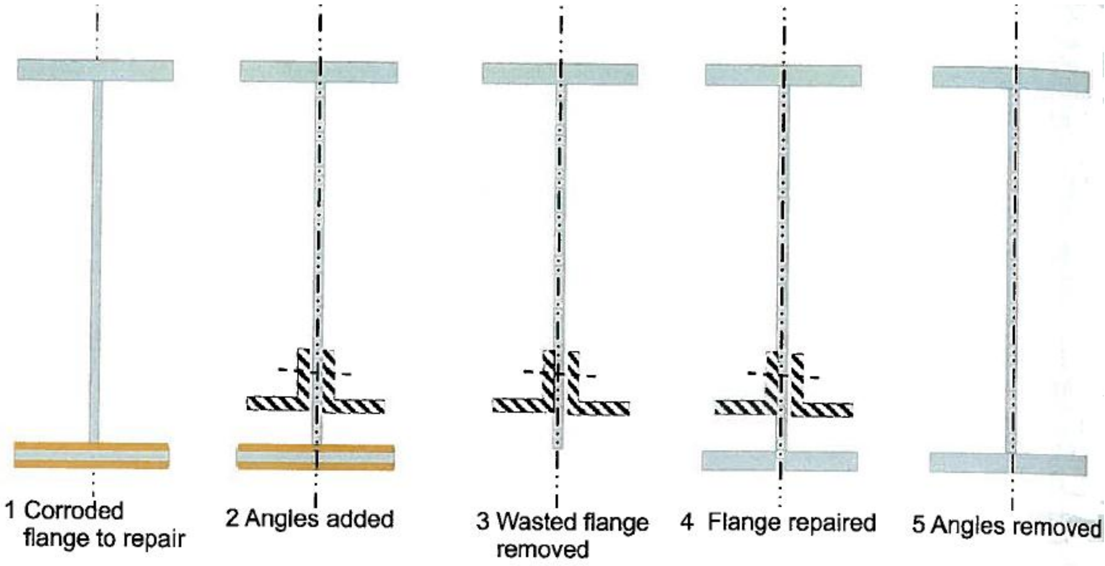


Figure 6.3 Corroded bottom flange is replaced [1].

6.5.3 Refurbishment of riveted connections

An example of a deteriorated riveted connection can be seen in Figure 6.4. These two pictures are taken from a former railway bridge which is currently used as a footbridge. The deterioration showed in Figure 6.4 is located on the support where the trusses are connected [1].

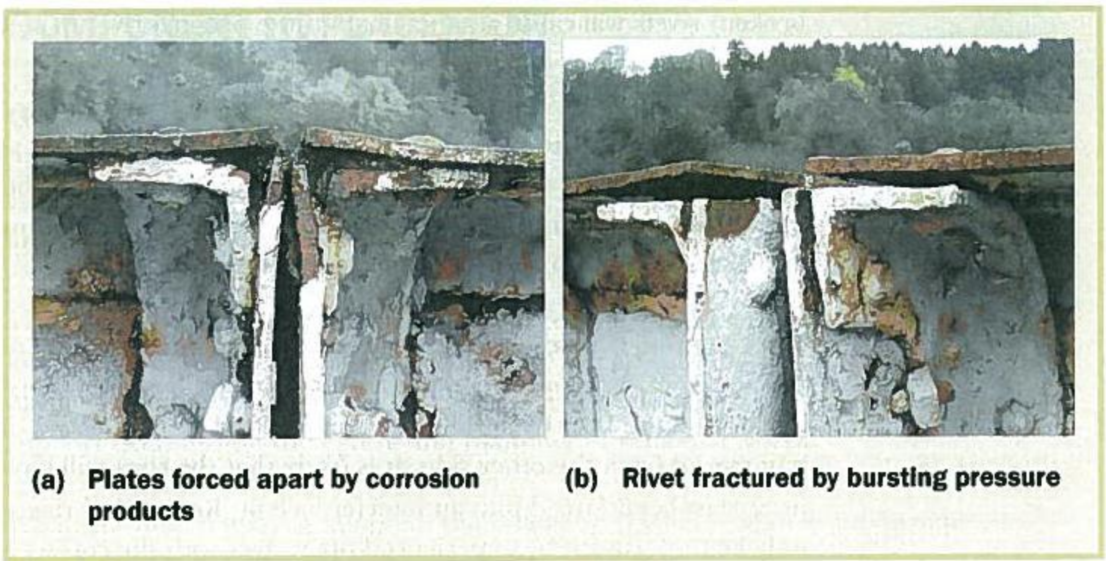


Figure 6.4 Deteriorated rivet connections [1].

In this case, exposure to water, caused corrosion formation between the plate faces and stiffener angles, see Figure 6.4 (a). Consequently, the coating or protective treatment was broken down and corrosion occurred in the joint. The corrosion product has greater volume than the original metal and this caused a gap opening between the plates. Figure 6.4 (b) shows the fracture of the rivets .In this example the rivets were replaced by HSFG bolts after the corrosions and other contaminations were removed [1].

When changing the existing rivets, single headed rivet is heated and then the shank is placed into the hole in the structure from one side see Figure 6.5 [1].

Figure 6.5 Heating of the rivets and introducing the holes [15].

Figure 6.5 and 6.6 show how the second rivet head on the other side is shaped by pressing and hammering. The connected plates are drawn together by cooling effect of the rivets [1].

Before replacing the rivets in connections, the engineer should understand the load paths across and in the connection. Today, old rivets are not replaced by new rivets due to complexity of application. Usually, HSFG bolts or other fasteners such as Huck bolts or tension control bolts (TCB) are used to replace rivets [1]. The noise from hammering should be considered before the operation.

Figure 6.6 The second rivet heat is shaped [15].

Flame cutting and drilling of the rivet are other methods to remove rivet heads. Flame cuttings can damage the parent material and therefore this method should be performed under full control of experienced staff. Drilling method is also considered to be a time consuming method [1].

6.5.4 Repair of deformed members

Structural members in bridge structures can be damaged by accident by vehicles or manufacturing in the factory etc. Depending on the limitation and codes, the deformed structural member can be repaired by mechanical and thermal repair methods [2].

The mechanical repair is most common repair method and the process is carried out by application of external loads to the deformed member. The deformed member is loaded in opposite direction to the deformation. The use of this method causes a loss of the yield strength in the member. There are additional requirements to national and international codes and recommendations for the mechanical method, namely [2]:

- Under temperature -20° the mechanical straightening is not permitted
- The maximum external forces should be continuous about 15 minutes in the last step of the operation
- No tolerance for cracks and defects, caused by mechanical repair
- First bending deformations should be removed, then the torsional deformations

The location and size of the deformation and the depth of the plate are decisive in determining tools and equipment to be used in the repair operation. For example, in case of small deformations and thin plates, hand tools are sufficient to use, see Figure 6.7. For large deformations and members, hydraulic jacks, power winches, chain blocks, pulley blocks, rigging screws etc. can be used to create external force [2].

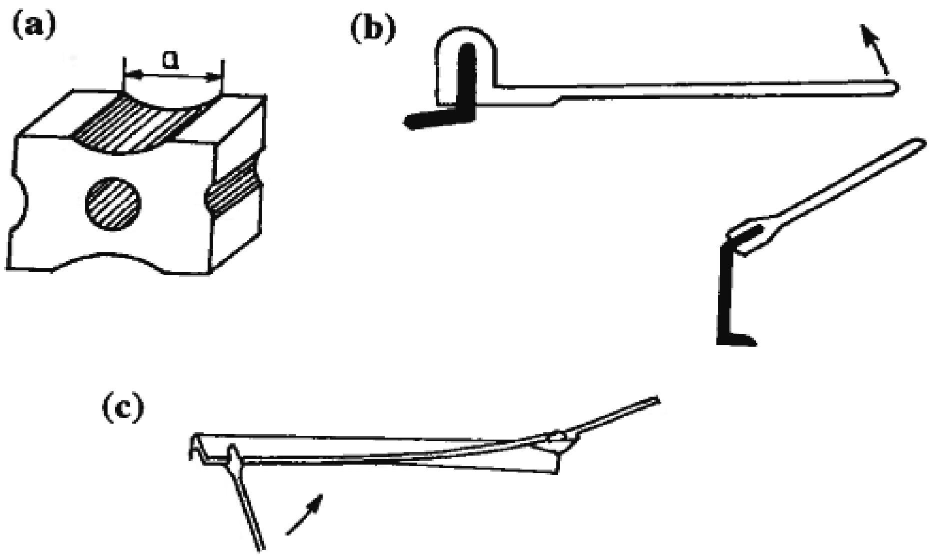


Figure 6.7 Hand tools, a) anvil, b) single-arm lever with grip, c) with two levers [2].

Figure 6.8 shows how a structural member in a truss chord is straightened by hydraulic jack [2].

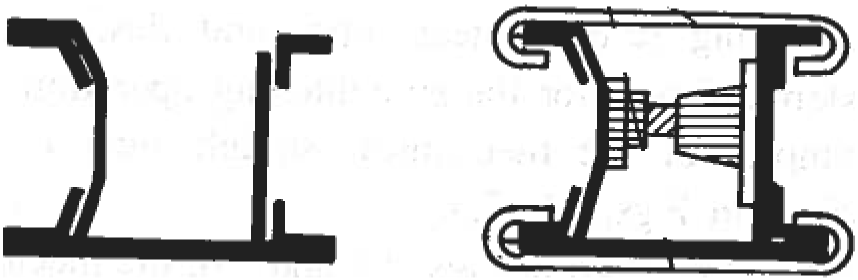


Figure 6.8 Straightening of a member by hydraulic jack [2].

Another example for application of hydraulic jacks for straightening is shown in Figure 6.9. In this case, a concave and convex diagonal member is straightened [2].

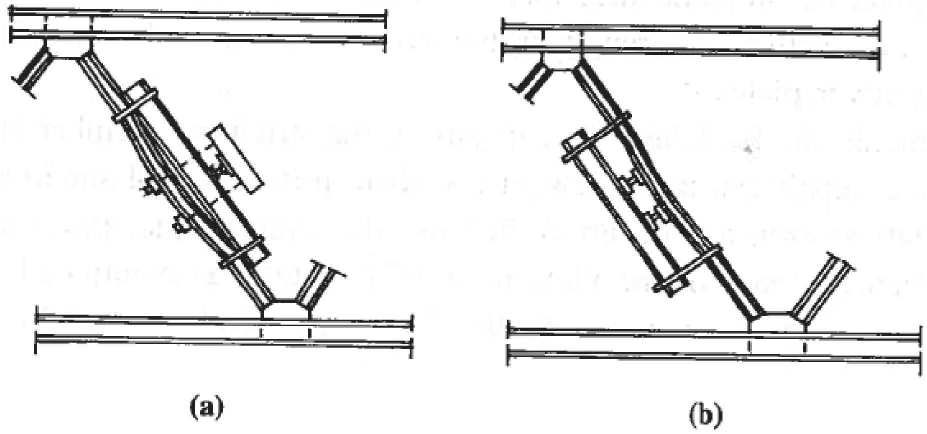


Figure 6.9 *Straightening by hydraulic jack [2].*

In thermal (heat) straightening process, damages such as distortion and buckling in wrought iron and steel members can be repaired [1]. This method is based on using heat (under control) to the plastically deformed component. Heating and cooling are repeated and the deformation is gradually straightened. There are different damage classes which have different behaviours and properties [2] [11].

Category S:

Bending about strong axis in built-up or rolled sections see Figure 6.10 [11].

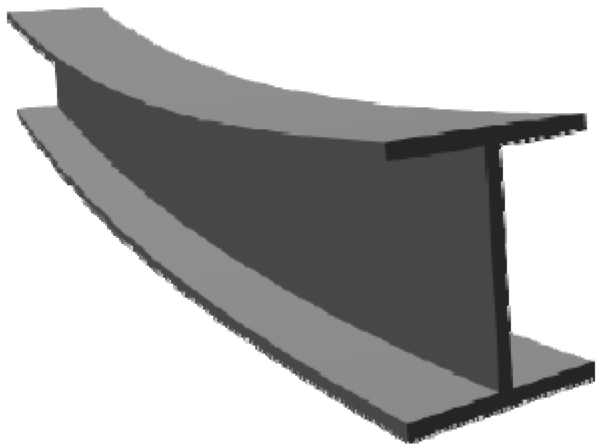


Figure 6.10 Category S [11].

Category W:

Bending about weak axis, see Figure 6.11 [11].

(a) Category W damage on a built-up double channel truss member. The damage was caused by a log falling from a truck on a bridge in North Louisiana.

(b) Category W damage to main girders during construction of a Louisiana bridge.

Figure 6.11 Category W [11].

Category T:

Torsional or twisting damage about longitudinal axis of the member is included in this category. Two examples are showed in Figure 6.12 and Figure 6.13 [11].

Figure 6.12 Category T, in this case the damage caused by hydraulic jack for experiment [11].

Category T damage on a composite bridge girder impacted by an over-height vehicle in Wisconsin.

Figure 6.13 Category T [11].

<u>Category L</u>:

This category includes damages such as local flange and web buckling, web crippling, bending in plates etc., see Figure 6.14 [11].

Category L damage showing flange buckles on wind bracing on Mississippi River Bridge in Greenville, MS.

Figure 6.14 Category L [11].

To heat the deformed material, oxy-acetylene torches are used. There are different heating methods, including [2];

- Spot heating
- Vee heating
- Rectangular heating
- Line heating
- Edge heating

Spot heating is applicable to small and round areas of the structural component. Limitation of the diameter of the area can be found in national and international codes. For example in some countries D is [2]:

$$D = 8g + 10 \text{ mm}$$

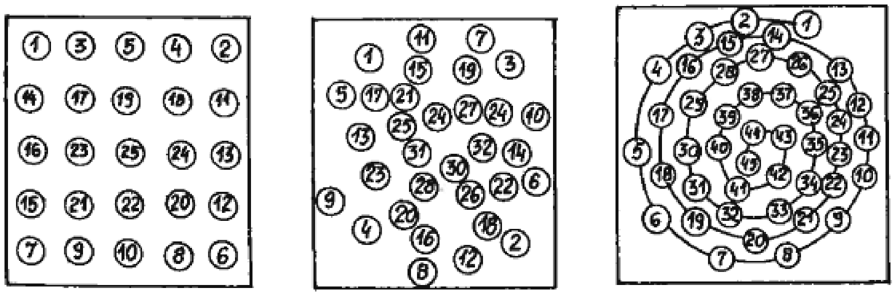
$$D \leq 25 \text{ mm}$$

$$D \leq 4g,$$

Where g is the depth of a deformed plate and g is in [mm].

The applied temperature on the steel plates should be between 650 and 723 °C [2]. The temperature should not be more than 723°C because of austenite starts at this temperature [2].

Spot heating is mostly used to remove wavy deformations and it can be achieved by row heating, radial heating or spiral heating as shown in Figure 6.15 [2].



(a) (b) (c)

Spot heating applied to wavy deformed plates: **(a)** row heating, **(b)** radial heating, **(c)** spiral heating. Numbers in the circles indicate the sequence of heat operation.

Figure 6.15 Different ways of spot heating [2].

Regardless of spot heating method used, there are a few general limitations. Start point of the heating is 100 mm from the plate edge, heating should be started from each spot center and no place should be heated two times [2].

Vee heating is mostly used to straighten bulging or bends in the steel members made of welded or rolled sections (damage category S) [2] [11]. In Figure 6.16, Vee and rectangular heating are shown [2].

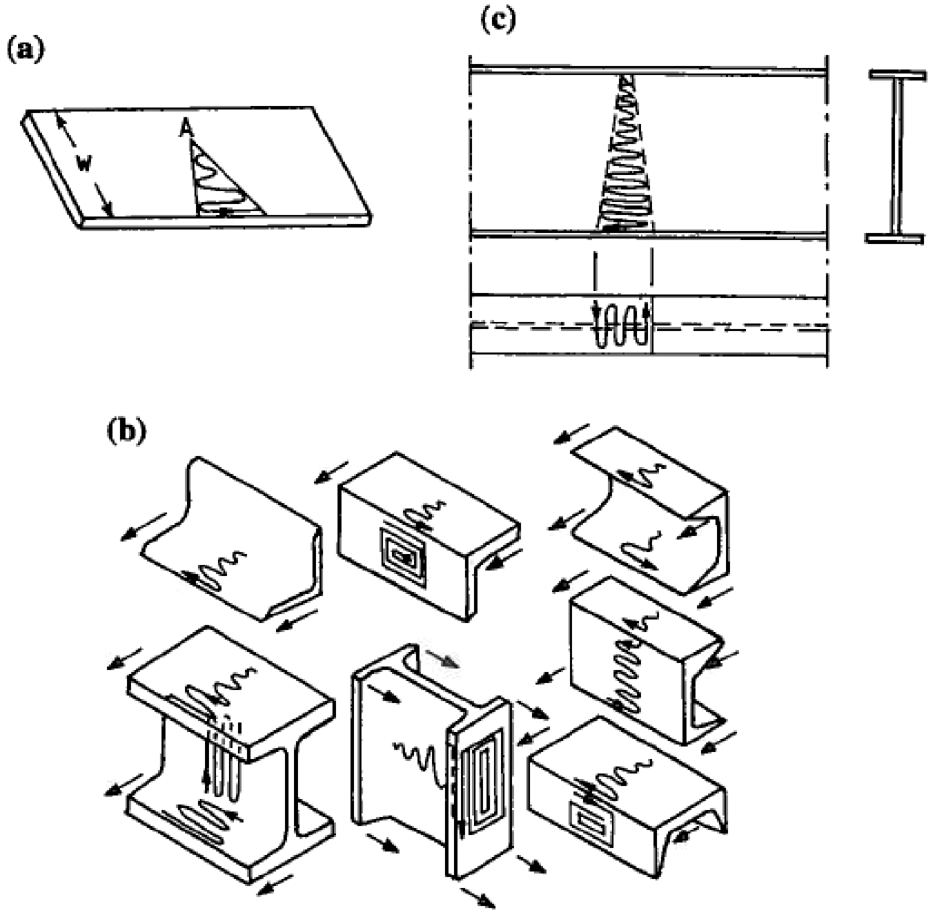


Figure 6.16 Vee and rectangular heating showed [2].

Some limitations for this method are:

- Start point of the Vee heating is tip of the Vee, see Figure 6.16
- The direction and process of the Vee heating as shown in Figure 6.16
- No stop and start points, meaning the operation should be done continuously

In Figure 6.17, Vee heating of bottom flange of a plate girder is shown in sequence [2].

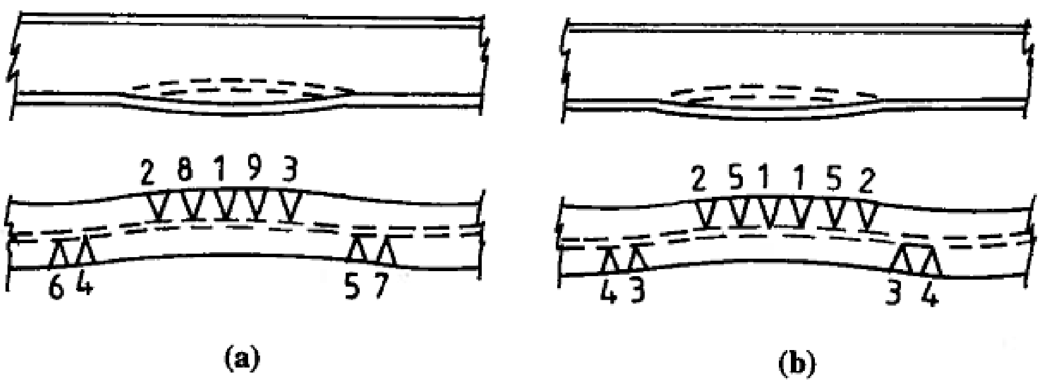


Figure 6.17 Sequence of the vee heating a) by one operator, b) by two operators [2].

Rectangular heating is mostly used to complete the Vee heating see Figure 6.17 [2].

Line heating is used to repair bended plate about weak axis and also bulging in the web. A bulging in the web can be repaired by applying the straight line heating on the convex side of the deformation [2] [11]. Figure 6.18 shows line heating process.

Figure 6.18 Line heating in the web [11].

Edge heating is mostly applicable to slender and very thin-walled members. In this process, the total length of the beam might be edge heated, see Figure 6.19 [2] [11].

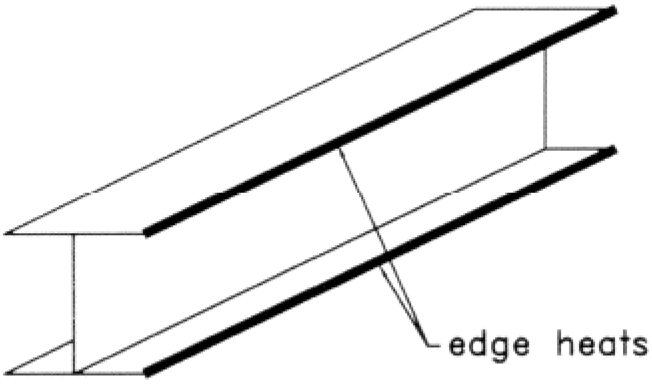


Figure 6.19 Edge heating [11].

In thermal (heat) straightening no temporary supports are required and full road closures are not needed [1]. Due to difficulties of this method staff with competence and experience is required [2].

6.6 Strengthening of members

When an extra load carrying in addition to original design value is needed, a bridge structure should be strengthened. Some reasons for strengthening are [3] [5];

- Underdesign
- Faulty construction
- New design codes
- Deteriorations e.g., corrosion or fatigue

- Increase of vehicle load and intensity
- Additional traffic lanes

6.6.1 Strengthening by modifying load paths

Some information about modifying load paths or reduction of load effect was mentioned in section 1 of this Chapter. The main idea behind this method is to divert the load from a critical region to other regions. This could be achieved by stiffness modification of the structure by for example adding plates, bracing or extra elements to the structure.

6.6.2 Strengthening by adding material

General

This method is based on adding extra plates to the existing bridge by welding, bolting or riveting. Adding plates is a common and can be applied to variety of bridge members.

During analysis phase, there are some issues that should be considered since they can influence the strengthening process, namely [2]:

a) For which load, the strengthening of the structure or the structural component is performed? For instance dead load and live load or only live load

b) Type of the dominant load in the critical component? Compression, tension, bending or torsion

c) Effect of the strengthening to the structure? E.g. effect of the redistributed internal loads on the structure

d) Connections? By welding, bolting, riveting

e) Accessibility of the component?

It is not common to strengthen the structure for live and dead loads because of the structure requires unloading by temporary supports before the process starts. This operation is not easy and relevant calculations are required [2].

Typical strengthening of the bottom flange of a girder is showed in Figure 6.20. In (a) and (b), horizontal plates, in (c) two vertical plates and in (d) two triangular plates are added to the component to increase the capacity [2].

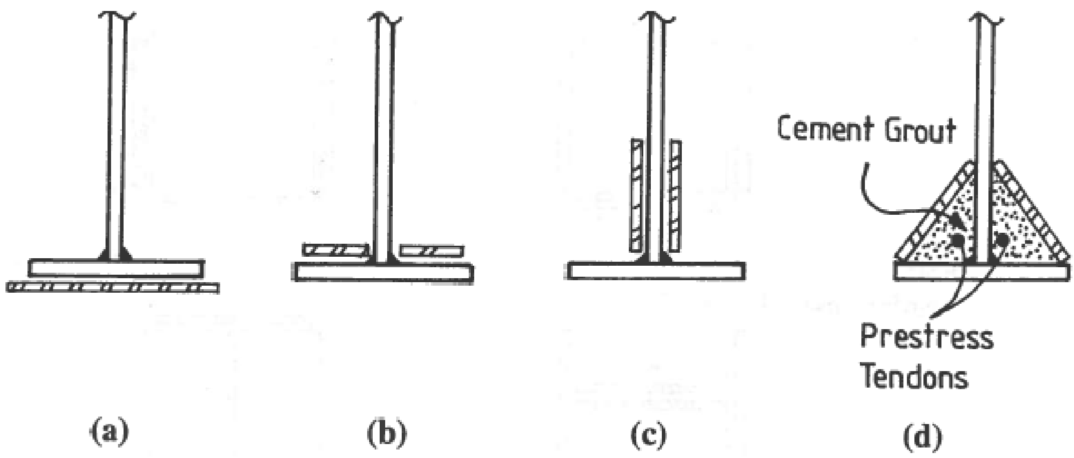


Figure 6.20 Strengthening of the bottom flange [2].

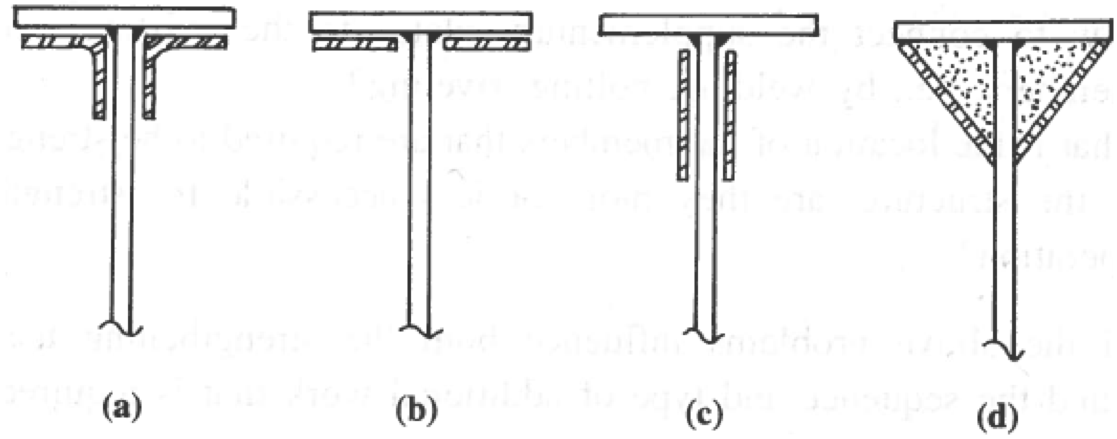


Figure 6.21 Strengthening of the top flange [2].

Strengthening of the top flange is often not an option due to overlying deck, example of top flange strengthening is illustrated in Figure 6.21 [2].

Figure 6.22 shows how the cross-section of a truss is increased by welding extra material [2]

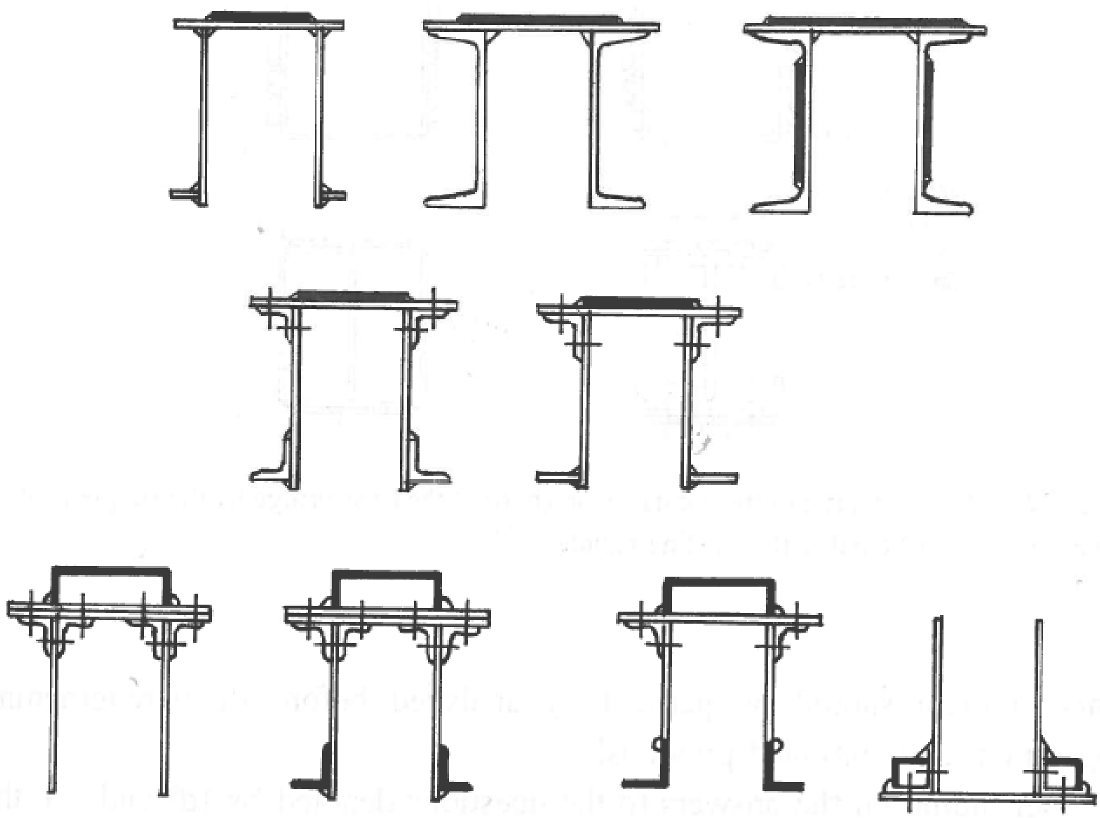


Figure 6.22 Strengthening by adding materials [2].

Redistribution of the internal force should be carefully analysed. Application of this method might result in need additional members or strengthening of other members. Two reasons are behind adding additional plates and sections to existing members: increasing the static strength and fatigue strength [1].

<u>Improving static strength:</u>

When a component of the structure is stocky and there is no risk for buckling, the extra material can be added to the member to increase its static strength. Analysis of the component and check of the influence on other components should be done accurately. Typical girder strengthening, top hat strengthening and pier strengthening are showed in Figure 6.23, 6.24, 6.25, respectively [1].

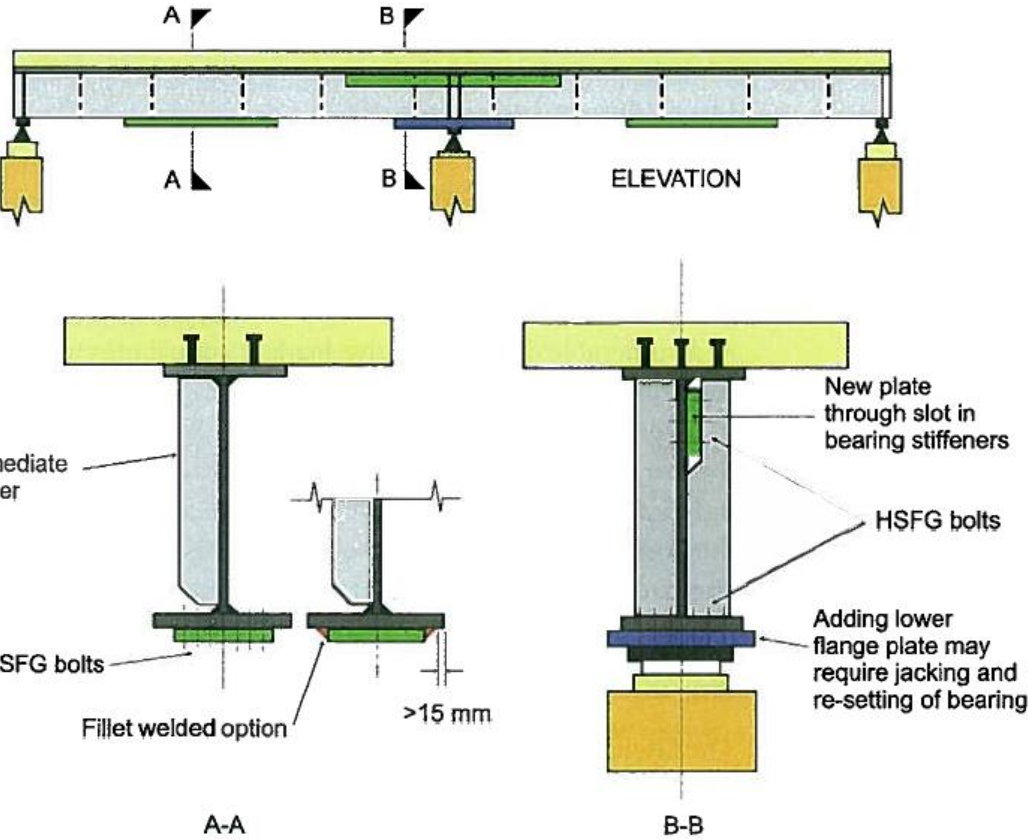


Figure 6.23 Girder strengthening [1].

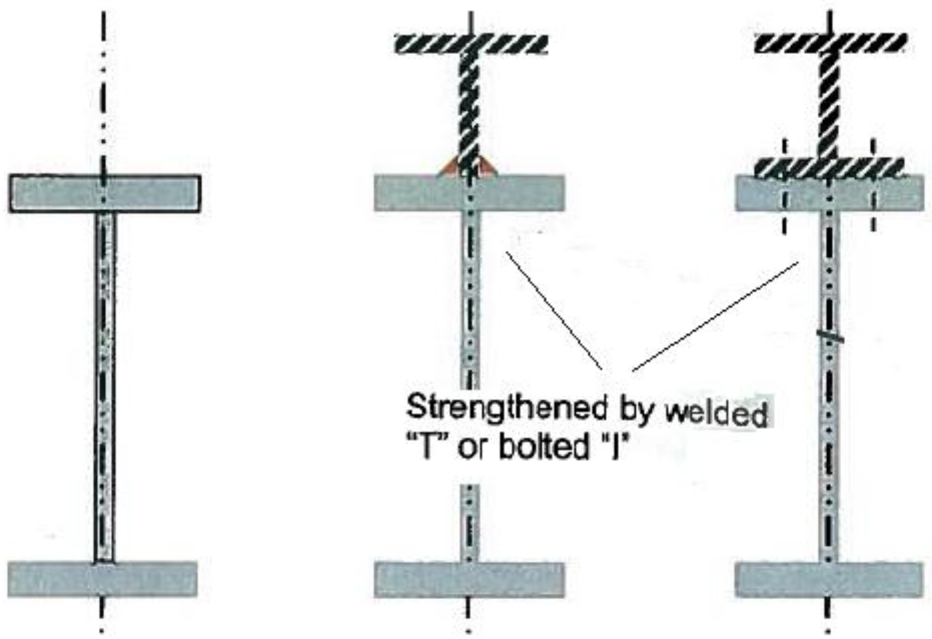


Figure 6.24 "Top hat" strengthening [1].

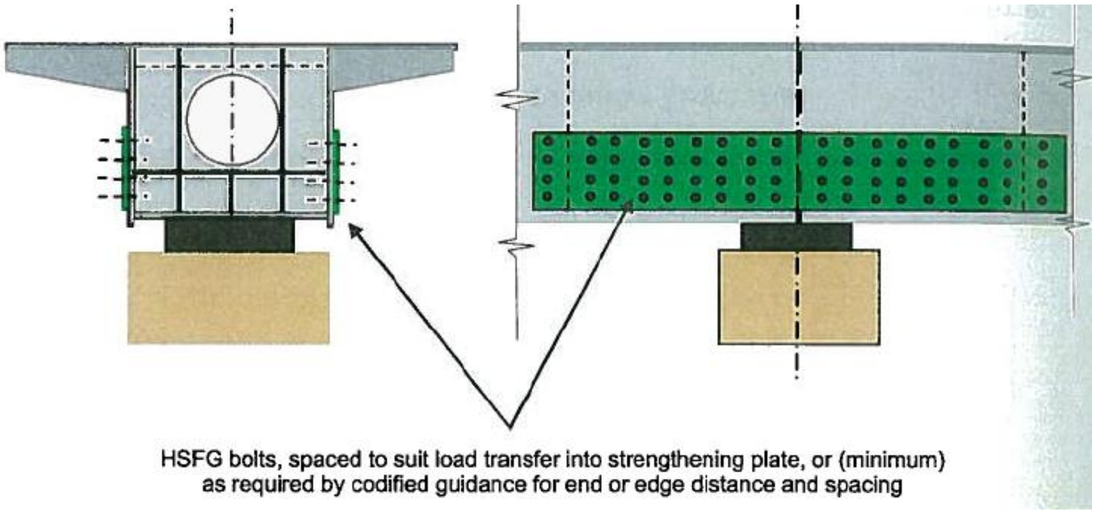


Figure 6.25 Pier strengthening for box girder [1].

Improving fatigue strength:

A sensitive component that is loaded repeatedly reduces the fatigue life of the structure. Addition of the extra material reduces the stress range in the sensitive component and thus increases the fatigue life. An example is shown in Figure 6.26 [1].

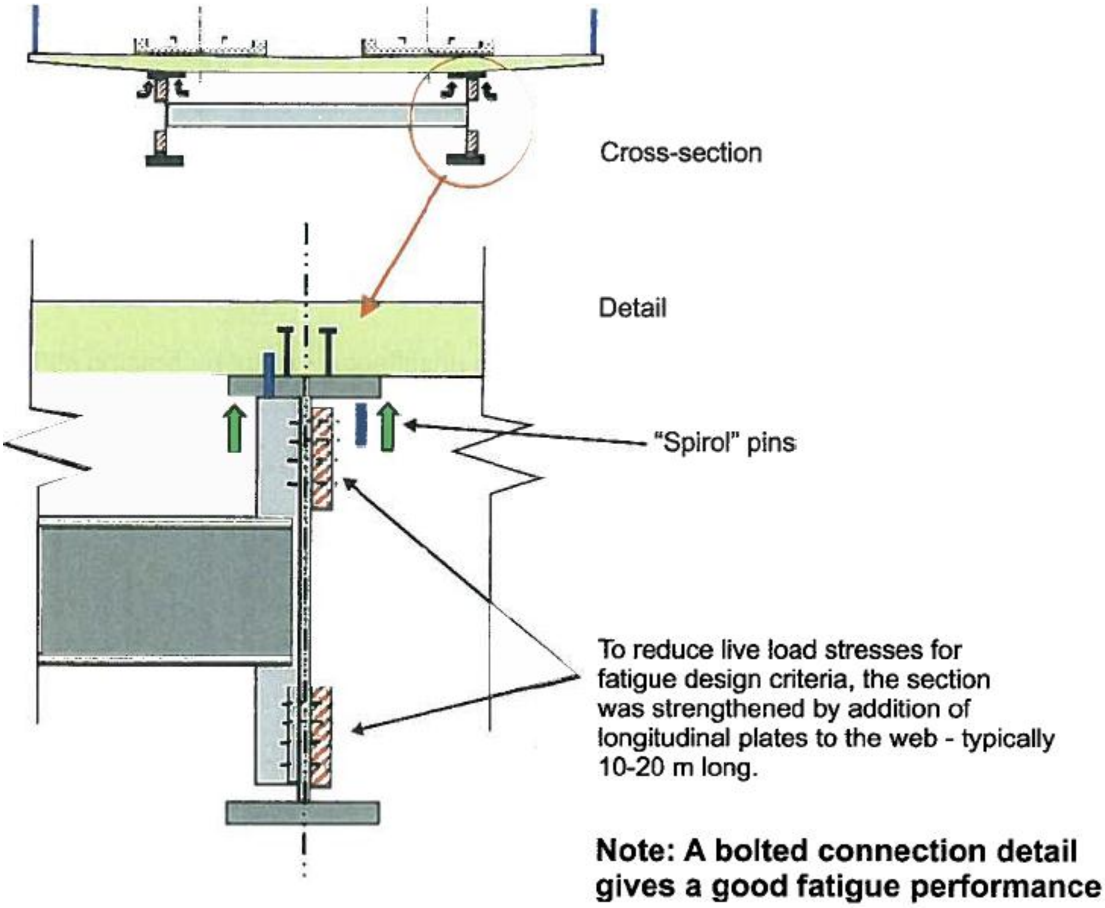


Figure 6.26 Strengthening, Docklands Light viaducts [1].

Stiffening

Buckling is a common problem for slender structural members that are subjected to compression. In this situation the buckling capacity is decisive, compared to yielding capacity. This problem can be solved by addition of stiffeners to the regions where the buckling capacity is lower than yield capacity. Some examples are:

- Adding stiffeners to the web panels on plate or box girders
- Adding longitudinal stiffeners to web panels between vertical stiffeners
- Increasing the thickness of the web or flanges
- Reducing the effective buckling length strength by adding new restraints

Figure 6.27 shows additional stiffeners in location of the support zone [1].

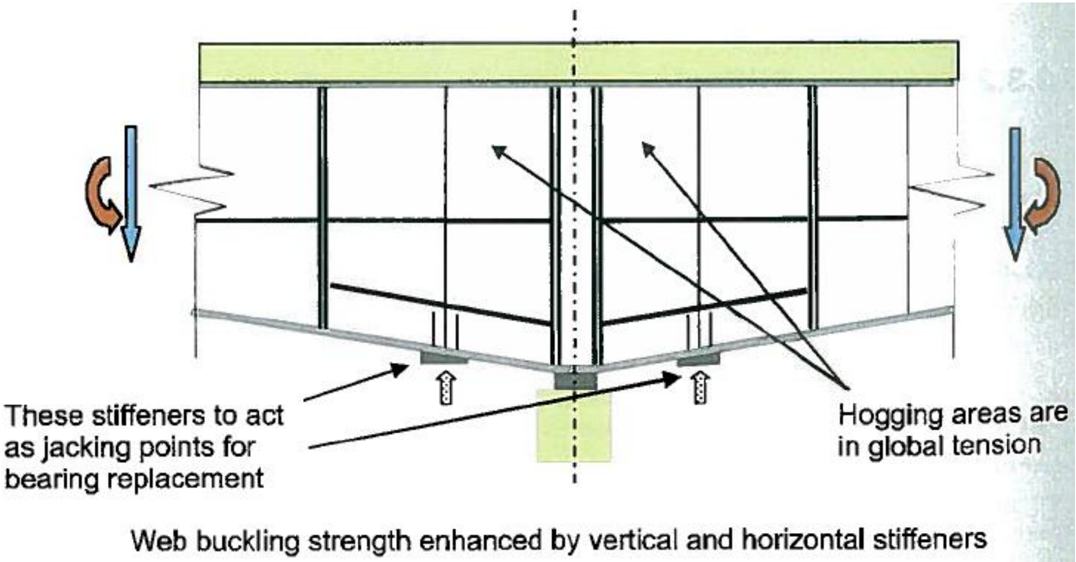


Figure 6.27 Additional stiffener in support zone [1].

Connections

<u>Welding:</u>

Weld is the most common and practical joining method to add extra material to an existing member. For this purpose, firstly, the weldability of the existing component should be checked and design of the weld should be carefully carried out. In the design process, increase in temperature in the existing component should be considered. Another issue to be accounted for is the shrinkage of the weld. Cracks, distortion of the neighbouring details and distortion of the whole structure are some examples due to weld shrinkage, see Figure 6.28 [1].

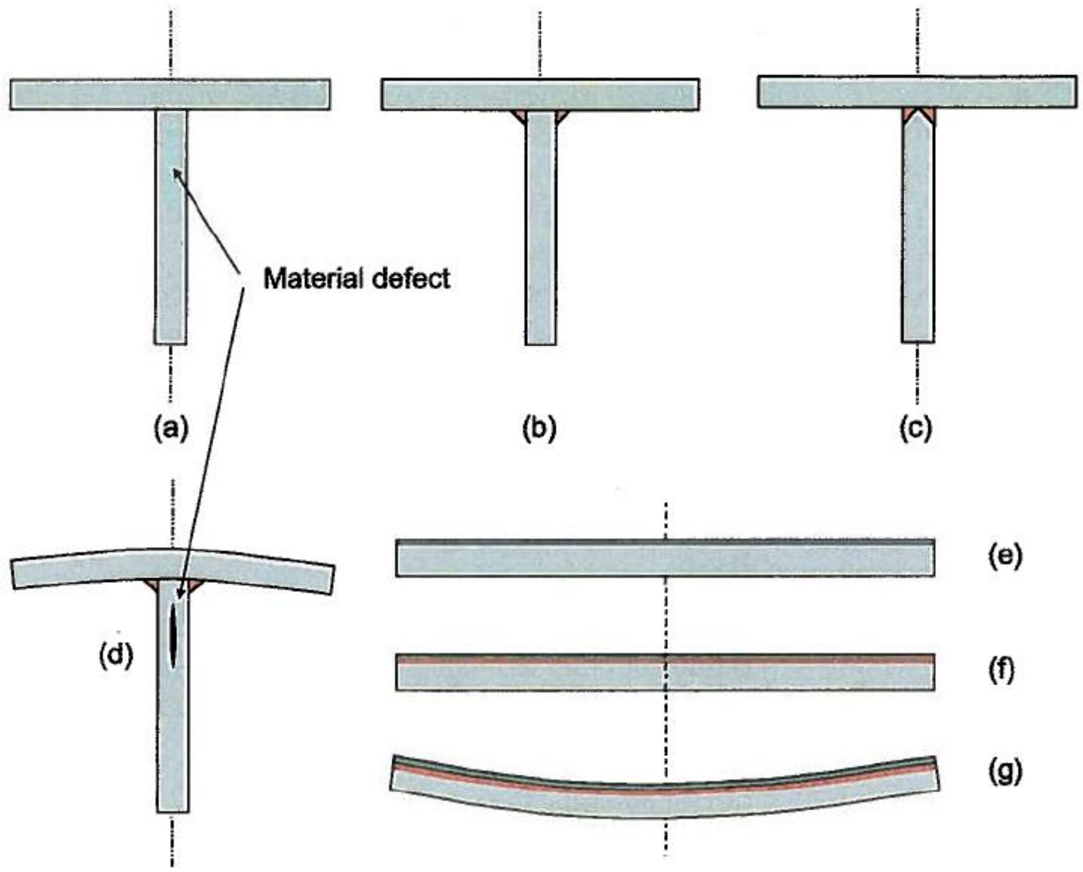


Figure 6.28 Illustration of Weld shrinkage [1].

In addition to abovementioned damages, there is a risk of lamination in the parent material to which the supplementary material is welded. This damage can be inspected by non-destructive testing, e.g., ultrasonic testing [1].

Attention should be paid to the following issues when using welding technique:

- Effect of heating from the welding process on the existing component
- Risk of buckling or distortion in slender components near the strengthening place
- Fatigue prone details
- Safety of working environment
- Accessibility
- Weld protection from the weather
- Position of the weld: downhand (preferable), vertical (allowable, but slower), overhead (to be avoided if possible)

<u>Bolting:</u>

Another alternative for joining is bolted connection. Bolts require more space compared to welds but preparation work is easier. Two types of bolts are common in structures [1].

1. Black bolts with strength grade 4.6 or 8.8

2. High strength friction grip (HSFG) bolts with strength grade 8.8 or more. Installation of HSFG bolts is showed in Figure 6.29.

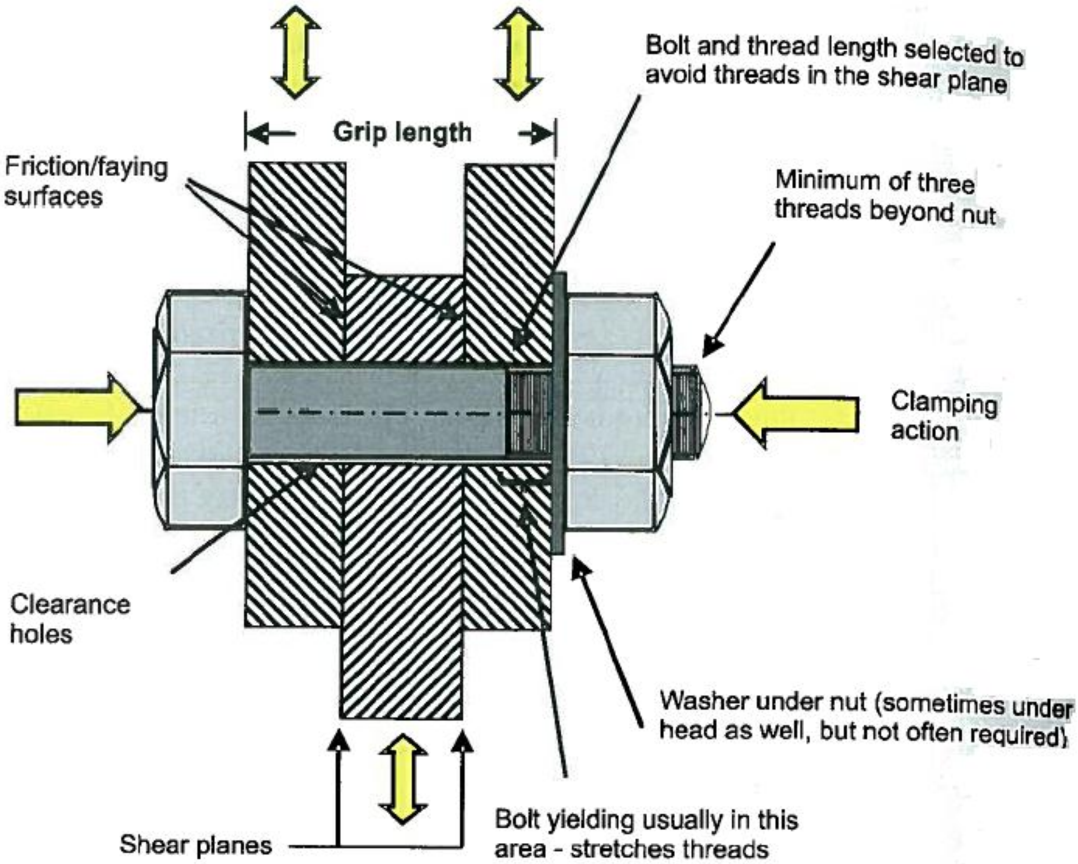


Figure 6.29 Process of HSFG bolts [1].

Black bolts can be installed directly and tightened with a wrench, but the HSFG bolts have to be designed and the torque is needed to be determined. Preloading of the HSFG bolts creates a clamping action on the joint and which improves the behaviour of the connection under fluctuating loads, e.g., vibrations. Bolts are generally installed in the holes with larger diameter than the bolt. This means that there is a risk of slippage between the plates of the connection. Clamping effect of HSFG bolts prevents the slippage. Therefore, for strengthening works the preloaded bolts and slip avoided connections are considered. For bridge strengthening HSFG bolts are recommended [1].

Installations of the HSFG bolts can be done by several methods, two of them are included in this report including part-turn method and torque control method, see Figure 6.30 [1].

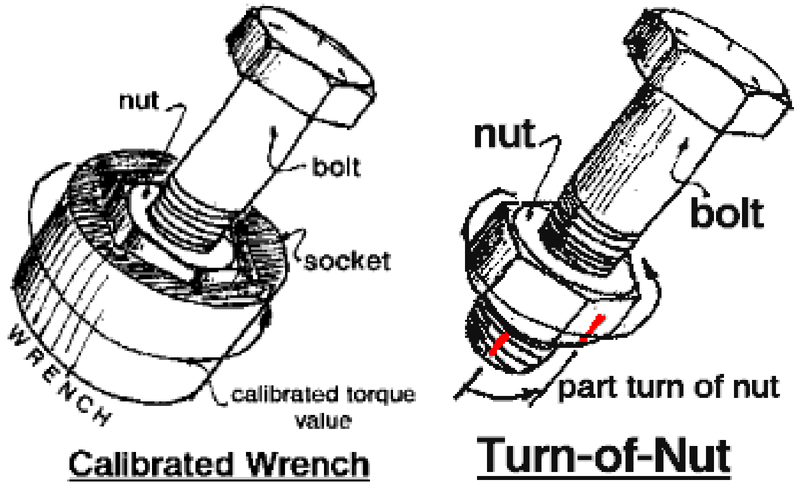


Figure 6.30 Different types of the bolting for HSFG bolts [13].

In the first method, after snugging the joint, the nut and the bolt shank will be marked, showed on the right side in Figure 6.30 above. Then a specific rotation is applied on the nut and bolt which depends on the length and diameter of the bolt. This method is not recommended when the steel surface is coated in case of thick paint layer or hot dipped galvanized zinc [1] [13].

In the torque control method, a special wrench with toque is used. The nut of the bolt is tightened until predetermined torque is reached [1]. Disadvantage of this method is that no torque table is available and in some countries this method is not permitted, for instance in Canada [13].

Reuse of the HSFG bolts is not recommended due to yielding of the bolt material. Threads of the used bolts usually show sign of stretching [1].

6.6.3 Strengthening and repair using Fibre-reinforced polymer composites

Strengthening of the structure or component can be performed by bonding fibre reinforced polymer, FRP, composites to them. FRP composites consist of three main constituents, namely [1]:

- High strength fibres (carbon, aramid or glass)
- Polymeric matrix
- Additives

FRP bonding could be used to increase [1] [5]:

- Axial tensile capacity
- Flexural capacity
- Shear capacity
- Fatigue life
- Stiffness

Comparison between conventional repair and strengthening methods and FRP bonding

Conventional strengthening and repair techniques usually involves adding additional plates to weak parts and sections in the structure by means of bolting, welding or riveting. These methods are often labour intensive and costly. The enlargement of the cross-section results in additional dead-load in to the structure and affects the serviceability of the structure. During the strengthening operation, traffic control or closure is usually required. Supplementary plates require uplifting by winch or crane. Bolted or riveted connections require drilling in the existing components and the holes result in reduction of the cross-section and increase in stresses [5].

Some advantages of FRP composite are high strength to weight ratio, high elastic modulus, corrosion resistance, fatigue resistance and good durability [1] [5]. FRP composite is more attractive when access to the critical section is difficult. Time of the application of the FRP composite is less than the conventional repair techniques [1].

However, there are some limitations regarding application of this method including [5]:

- Insufficient design codes for adhesive joints
- Dependency on workmanship's skills
- A lack of knowledge about the long term performance

Properties of reinforcing fibres

Table 6.1 shows properties of aramid, carbon and glass fibres. Carbon fibres are available in three types, high strength (HS), high modulus (HM) and ultra-high modulus (UHM). Carbon fibres have low thermal expansion but high electrical conductivity [4].

Table 6.1 Material properties of carbon fibre, aramid fibre and e-glass fibre [5].

	Carbon fibre			Aramid fibre	E-glass fibre
	High-strength (HS)	High-modulus (HM)	Ultra-high modulus (UHM)		
Modulus of elasticity (GPa)	230–240	295–390	440–640	125–130	70–85
Strength (MPa)	4300–4900	2740–5940	2600–4020	3200–3600	2460–2580
Strain to failure (%)	1.9–2.1	0.7–1.9	0.4–0.8	2.4	3.5
Density (kg/m^3)	1800	1730–1810	1910–2120	1390–1470	2600
Coefficient of thermal expansion (parallel to fibre), (10^{-6}/°C)	-0.38	-0.83	-1.1	2.1	4.9

Aramid fibres, also known as Twaron and Kevlar® in the market, have high strain at failure and high strength, see table 6.1. Glass fibres have the least strength but they are considerably cheaper than other fibres [4].

Adhesives

Phenolic adhesive was the first commercially available adhesive used for structural bonding in 1940s. Different bonding types are showed in Figure 6.31.

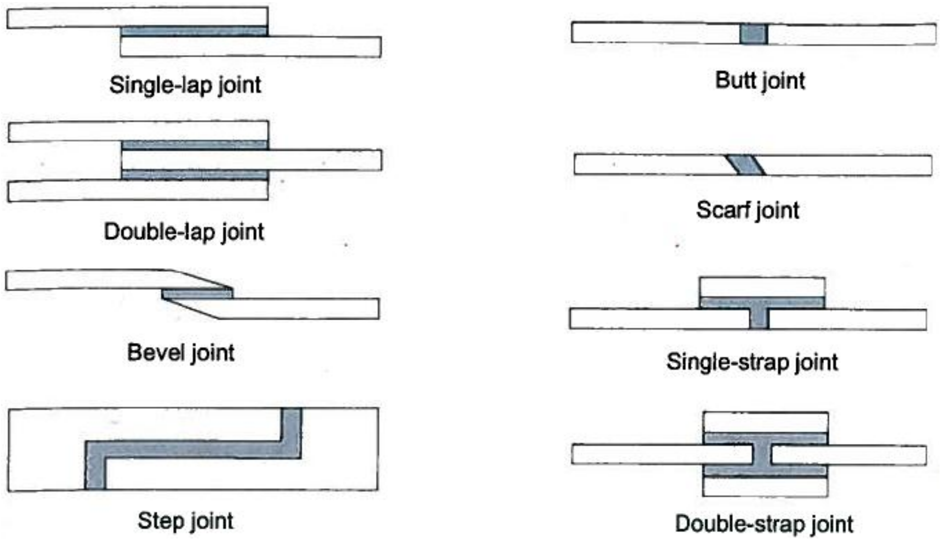


Figure 6.31 Different types of adhesive bonding [5].

Some advantages of adhesive technique compared to the other or joining methods (bolting, welding and riveting), are:

- Good fatigue resistance
- High stiffness
- Easy application
- Less time consuming and costly
- Application on dissimilar materials

FRP strengthening systems

<u>Prefabricated plates:</u>

There are two methods to produce the prefabricated plates, pultrusion process and preformed prepreg (pre-impregnated fibres) plates [1].

Pultrusion process is an automated manufacturing method, see Figure 6.32.

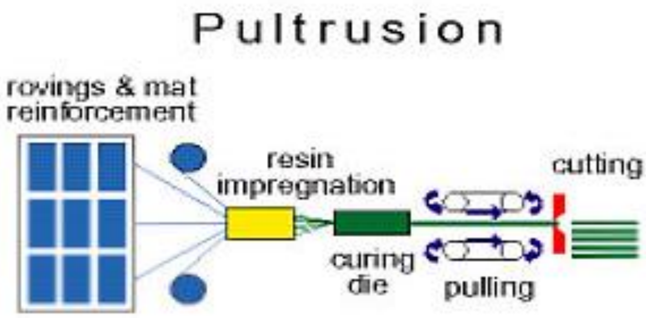


Figure 6.32 Pultrusion process [14].

In this method, firstly, resin is added to fibres by means passing them through a resin tank and then the fibres are put in direction along the length of the plates. To create the custom-built product, prepreg mats are applied to the plate by hand and then it is cured under high pressure and temperature. These plates are usually expensive due to the production process [1].

Wet lay-up systems:

In this system the mats of the FRP are applied to the structure manually and then FRP is impregnated with a liquid resin system. Consolidation of the FRP can be made by a vacuum bag by means of a film applied over the FRP composite. Then a polymer membrane is applied, to seal along the FRP edges and finally air is removed from the bag [1].

Resin infusion techniques:

This method is similar to the previous method. In this technique dry fibre mats are used. After application of dry fibre mats, a diffusion membrane and a vacuum bag are used over dry fibre mats. To apply the resin and impregnate the fibre mats, vacuum is used [1] [4].

In situ prepreg (pre-impregnated) lamination:

In this method, application of the pre-impregnated mats to the structure is completed and then a vacuum bag and electric heating blanket are placed over the mats. The resin is then cured at high temperature and under pressure. Production of composites with high strength and stiffness properties can be achieved by this method together with the resin infusion techniques [1].

Design issues

Adhesive joint analysis:

The load transfer from the substrate to the FRP laminate is achieved by the shear action in the adhesive layer. The magnitude of the shear and peeling stress at the end of the bond line in the adhesive layer is very high and this the failure in the joint usually starts at this location, see Figure 6.33 [1] [4] [5].

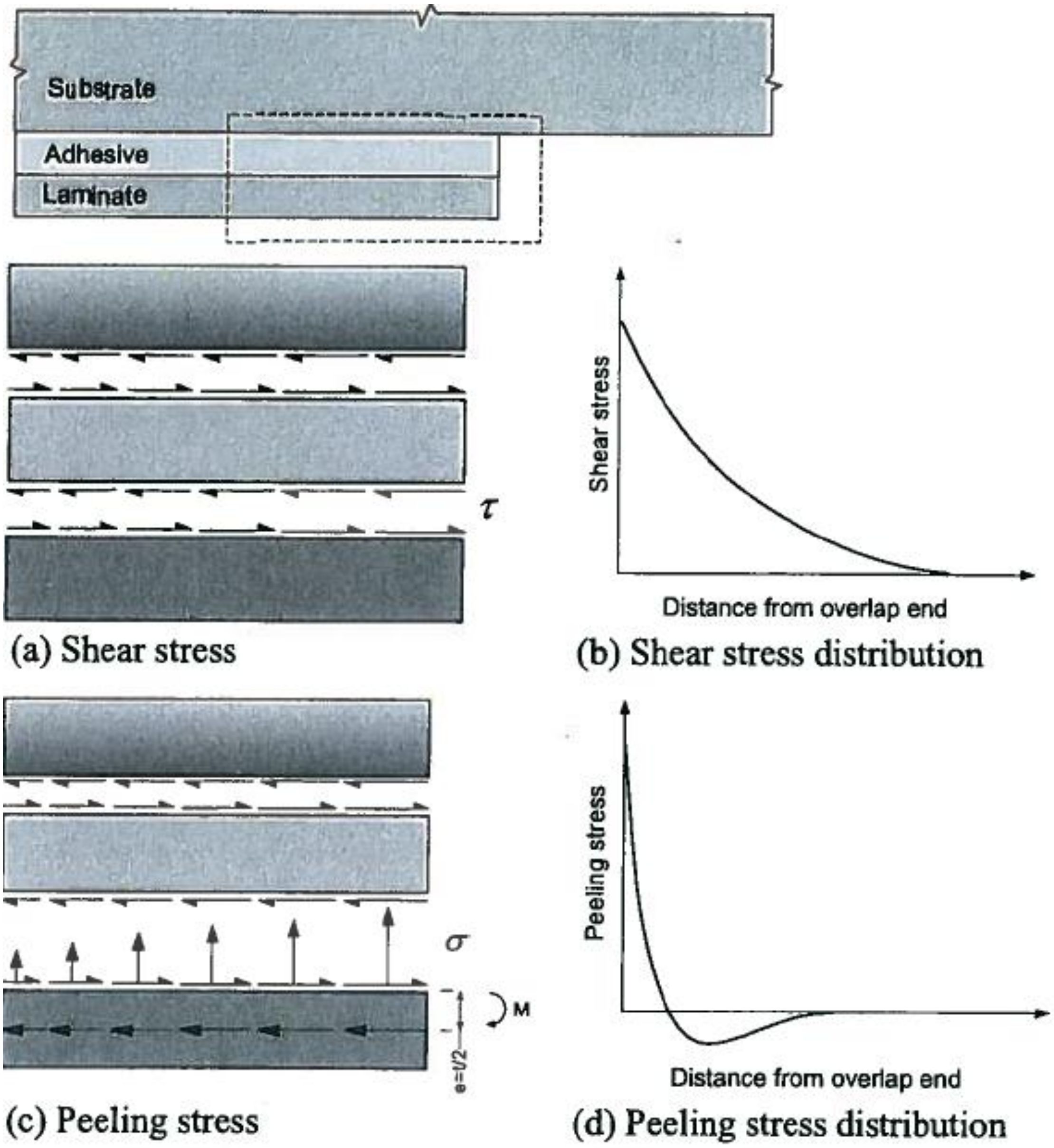


Figure 6.33 Illustration of shear and peeling stress [5].

Stress analysis is an important and difficult part of the joint design. Analytical and numerical analyses are useful to design adhesive joints and usually these two methods complete to each other. Table 6.2 shows a comparison between these methods [5].

Table 6.2 Comparison between the analytical and numerical methods [5].

Method	Advantages	Drawbacks
Analytical analysis	Simple Fast	Developed for specific configuration Considers ideal material response Returns results at specific locations
Finite element analysis	Provides more detailed results Treats complex geometries Considers complex material behaviour	Time consuming Requires expertise for modelling and interpretation of results Stress singularity and mesh dependence

Fatigue resistance, creep behaviour, fire performance are among issues which should be considered when using FRP bonding technique [4].

Thermal effects:

Table 6.3 shows thermal expansion coefficients for different constituents in an adhesive joint.

Table 6.3 Thermal expansion coefficients for steel epoxy and CFRP [5].

Material	Coefficient of thermal expansion (1-°C)
Steel	11-13 [10^{-6}]
Epoxy	50-55 [10^{-6}]
CFRP	~-1 [10^{-6}]

The difference in the thermal expansion coefficient might cause great shear stresses in the adhesive joints which are high enough to damage the joint. This issue should be considered when designing adhesive joints [5].

Corrosion:

Galvanic corrosion is an issue when two different metals are bonded together and a current flow is arose. Due to conductivity of CFRP laminates, this risk is increased. The risk of the galvanic corrosion can be reduced by careful application of adhesive joints and additional glass fibre layer between the CFRP and the substrate [1].

Installation

Installation of the FRP composite is generally explained in Table 6.4 and showed in Figure 6.34 [5].

Table 6.4 Steps of the FRP composite installations [5].

Step	Description
1	*Surface preparation.* To remove any dust, paint or contamination from the surface. The recommended method is sand blasting. Chemical cleaning of the surface after blasting using a suitable solvent, e.g. acetone, is performed.
2	*Application of primer.* To enhance the chemical bond between the adhesive and the metallic surface. The application of primer should take place shortly after blasting to prevent possible corrosion of the surface
3	*Preparation of laminates.* To enhance the bond between the laminate and adhesive layer. Some laminates are equipped with protective peel plies which should be removed before application. Otherwise, laminates should be prepared by slight sanding followed by cleaning with a suitable solvent, e.g. acetone
4	*Preparation of adhesive.* The adhesive should be prepared based on the manufacturer's recommendations. In the case of two-component adhesives, attention should be paid to the correct ratio and mixing speed.
5	*Application of the adhesive.* After curing the primer layer and completing steps 3 and 4, the adhesive is applied. Depending on the accessibility of the metallic surface, the adhesive can be applied to either the FRP laminate or the metallic substrate. Care should be taken in order to avoid defects in the adhesive layer, e.g. air bubbles
6	*Application of laminate.* After completing step 5, laminates should be carefully placed on the structure. It is important to complete this step before the pot life of the adhesive is reached. Care should be taken in order to keep the laminates and the adhesive layer clean during this step. Any displacement of the laminate during the curing time should be avoided

Figure 6.34 View of FRP installation [5].

Strengthening and repair of structures using FRP bonding technique is rather new and success of the operation depends on the quality of the application and experience of the staff.

Application of this method in the ambient temperature under 5°C is not recommended since the curing process stops at this temperature [1].

7 Case studies

To have a better understanding of remedial works, different types of remedial works are studied. This work is separated into two parts, called case studies 1 and case studies 2.

In case 1, different cases (about 39 cases) were investigated and 11 relevant/interesting cases of them were taken into account in this thesis. All cases were operated in UK and therefore information about their recommendations, codes and type of contract were neglected.

In case 2, a few bridges located in Sweden were investigated. The investigation concerned only painting and surface damages. The work contains briefly introduction of bridges and their damages.

7.1 Case studies 1

7.1.1 Midland Links—bearing stiffeners

The Midland Links Motorway is located near to Birmingham, UK and carrying M5 and M6 motorways [3].

Buckling of the webs of the universal beams and plate girders were investigated. A reasonable safety factor against buckling of the universal beam webs was noticed but in case of the buckling of the plate girder webs a limited safety factor was noticed [3].

Installation of strengthening: At the position of all plate girder ends, the stiffeners were added by welding but at the abutments stiffeners were added by bolting due to limited access [3].

Table 7.1 presents brief information about the strengthening operation [3].

Table 7.1 Information about strengthening operation [3].

Inspection before the operation	The height, inclinations and bow of the webs were measured.
Tests before the operation	Ultrasonic testing of the webs to detect laminations in the webs. Chemical analysis to determine the welding process. A mock-up was built to illustrate the access for welding process and checked strain on the shear connectors if welding process caused to unacceptable level of the strain, see Figure 7.1. Carbon content of the material was tested for welding process.
How fasten?	Fillet welding used and laminations were buttered.
Traffic	Reduction of the load from 45 to 25 units of HB (1 unit HB corresponds to 10 kN per axel) during the welding to avoid overstress due to reduced strength because of a hot steel. Welding process was

	performed during day.
Tests after the operation	MPI and ultrasonic testing on the welds to check defects. (Not many defects were found).
Time	Generally within the estimated time schedule. Grinding took more time than the contractor counted.
Economic aspects	Generally within the cost plan.
Problems during the operation	Fitting the stiffeners required careful supervision. At some places in the web, bows were detected and when the bolted stiffeners were tightened, could not help to straighten the bows. Epoxy mortar added to fill the gap.
Anything went badly?	No.
Changes needed if do again?	No main changes.

Figure 7.1 A mock-up was built before the operation [3].

7.1.2 Rakewood Viaduct—presstressing

Rakewood Viaduct is carrying M62 motorway and consists of six continuous spans see Figure 7.2. The maximum height of the Viaduct is 36m see Figure 7.3 [3].

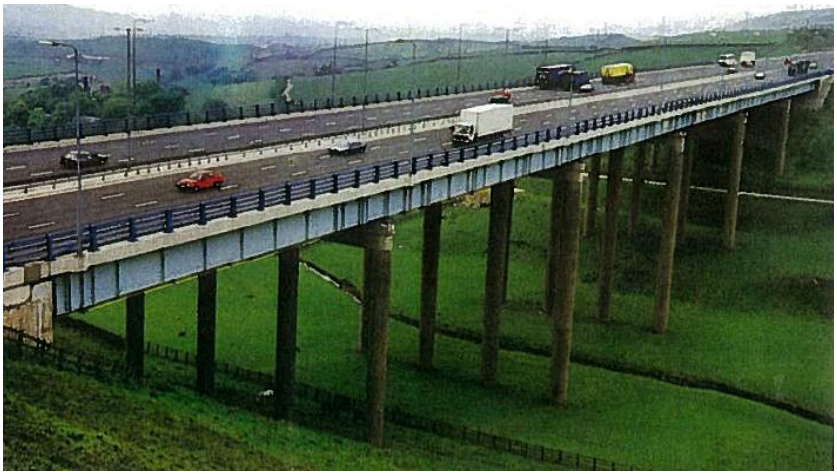


Figure 7.2 Rakewood Viaduct [3].

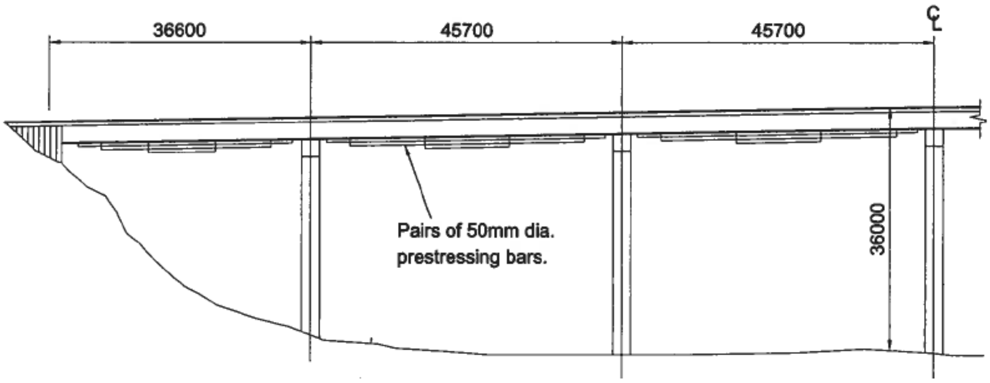


Figure 7.3 Maximum height of the viaduct showed [3].

Due to high wind and snow loads, ice in the winter and heavy vehicles on a sharp inclined way, the hardshoulder was changed to an additional lane. The bottom flanges of the main girders over the piers (where the hogging zones exist) were overstressed [3].

Strengthening work limited: The prestressing bars were added to reduce the hogging moments [3].

Installation of strengthening: The prestressing bars were used. The anchorage of the bars to the bottom flange was made by HSFG bolts see Figure 7.4 and to provide resistance to induced vertical forces, bearing stiffeners were added [3].

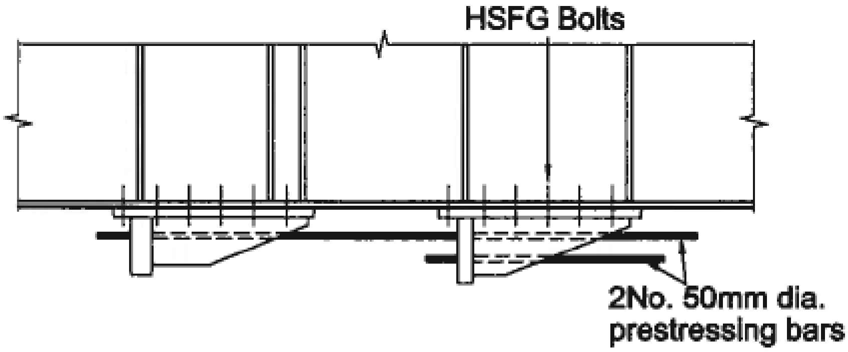


Figure 7.4 The anchorage of the bars by HSFG bolts [3].

Considered alternatives: To add plate in the bottom flanges was difficult due to the access of the critical regions [3].

Table 7.2 presents brief information about the strengthening operation [3].

Table 7.2 Facts of the strengthening process [3].

Inspection before the operation	Plate panel and stiffener stability were inspected. All welding checked.
Tests before the operation	Welding was tested and steel quality was tested to check brittle failure. This strengthening method was grounded on the earlier American practice.
How fasten?	By HSFG bolts and welding of high tensile steel from 1967.
Traffic	None management of traffic.
Tests after the operation	Permanent strain monitoring on the bars when stressed and afterwards.
Time	The operation done in estimated time.
Economic aspects	Within the cost plan.
Problems during the operation?	Contra-rotation and problems at the end bearings because of prestressing.
Anything went badly?	No.
Changes needed if do again?	High strength fibre composite tendons can be considered.

7.1.3 Huntworth Viaduct—steel box strengthening

The Huntworth Viaduct located in England, is carrying M5 and consists of 17 spans [3].

A crack was found in one of the rolling bearings and a piece of the roller had been cracked away see Figure 7.5. The roller bearings had a poor fatigue resistance and some of them were already cracked. The longitudinal stiffener along the bottom flange was also included to the strengthening operation due to the failed assessment of their bending moment. Shear capacity of the box girder web was insufficient (12 m from the support).

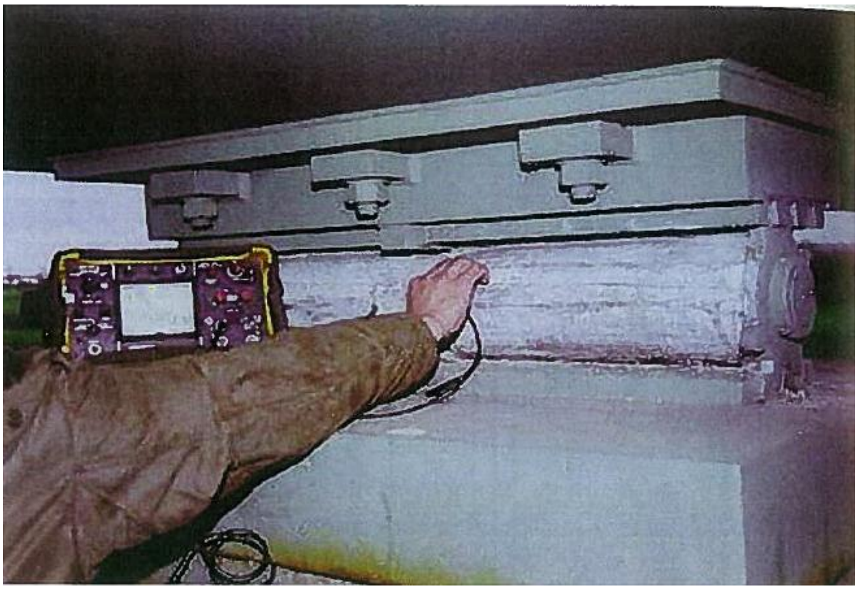


Figure 7.5 The damaged rolling bearing [3].

Strengthening work limited: A longitudinal stiffener of 9 m, along the bottom flange in location of piers was covered by concrete to reduce the effective length. In case of the shear capacity of the web, T-stiffener was added by bolting [3].

Installation of strengthening: Firstly, doubler plates were welded to the transverse stiffener at the jacking points and then the jacks were installed. Damage in the web welds were repaired after jacking. Instead of the rolling bearing, inverted pot bearings were installed see Figure 7.6 [3].

Figure 7.6 An inverted pot bearing in package [3].

Table 7.3 below is containing brief information about the operation of the strengthening [3].

Table 7.3 The facts about strengthening operation [3].

Inspection before the operation	Ultrasonic and MPI testing of welding.

Tests before the operation	For fatigue testing of the bearing acoustic emission testing was used under known load. Material testing to investigate the notch toughness.
Considered alternatives	Direct strengthening considered and concluded as expensive alternative.
How fit?	Firstly, the box was placed on the temporary support by jacking and then the bearing was replaced. The welds were repaired after removed stresses.
Time	The operation done in estimated time.
Economic aspects	Generally within the cost plan.
Problems during the operation?	The old bearings were not in the positions according to the drawings.
Changes needed if do again?	A pre-contract survey was recommended for the bearing positions.

7.1.4 Friarton Bridge--prestressing

The Friarton Bridge in Scotland is located over the River Tay and comprises nine-span box girder and an overlying lightweight concrete deck. The distance of the main span is 174 m [3].

Increase of the load on the bridge and change in the standards resulted in great tensile stresses in the top flange over the internal supports, see Figure 7.7. Safety factors against buckling of the webs and flanges were insufficient [3].

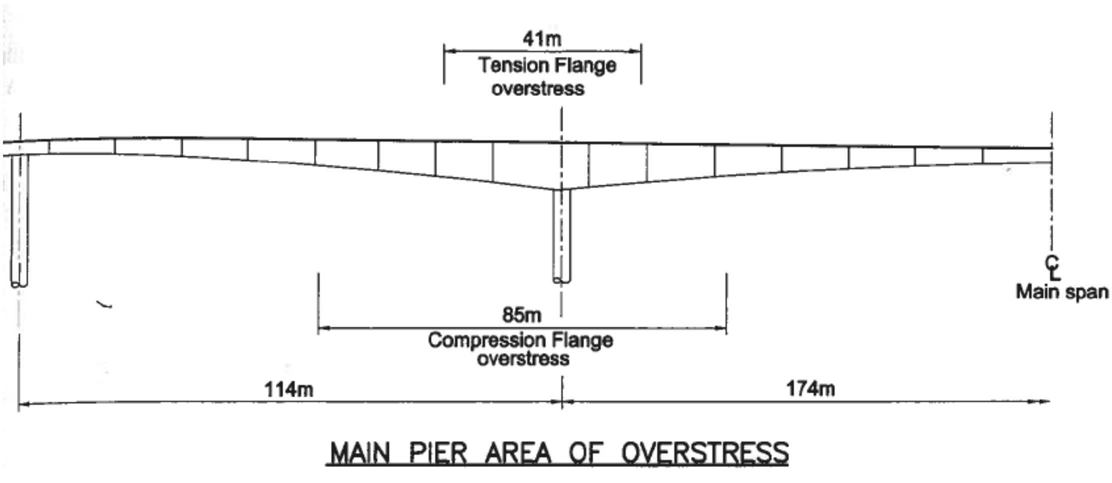


Figure 7.7 Overstressed tension and compression flange [3].

The shear studs capacity in strength and resistance against fatigue were insufficient [3].

<u>Strengthening work limited</u>: The result from dynamic monitoring showed that the slab and girder worked compositely. The utilizing factors were for the shear stud, 85% at Serviceability Limit State (SLS), 45% at Ultimate Limit State and 70% at maximum

fatigue stress. FE models had been done on the diaphragm and the result of stresses was under accepted level [3].

Installation of strengthening: To reduce tensile stresses over the supports, external prestress was used. For buckling of the webs and flanges, stiffeners were added [3].

Considered alternatives: Inclined cables were an alternative but due to a risk of overstressing of the bottom flange and the diaphragms it was excluded. Another alternative was that supplementary plates would bolt the bottom flange but this was excluded due to the heavy weight of the plates for installation by the maintenance gantry [3].

In case of the welding longitudinal stiffener to the top and bottom flange over the support zones where the hugging moments existed could increase the capacity but considered as ineffective (when the strengthening designed for only the live loads) [3].

Table 7.4 below is containing brief information about the operation of the strengthening [3].

Table 7.4 Information of strengthening operation [3].

Inspection before the operation	Inspection of the painting.
Tests before the operation	Vibration behaviour of the viaduct measured and crack degree of the deck measured.
How fasten?	Figure 7.8 shows the anchorage of the cables to plates of 150mm.
Traffic	Under strengthening one lane traffic was permitted and under welding no live load was permitted.
Tests after the operation	The vibration behaviour monitored during the strengthening.
Time	Yes, the operation done in estimated time.
Problems during the operation	No.
Anything went badly?	No.
Changes needed if do again?	No.

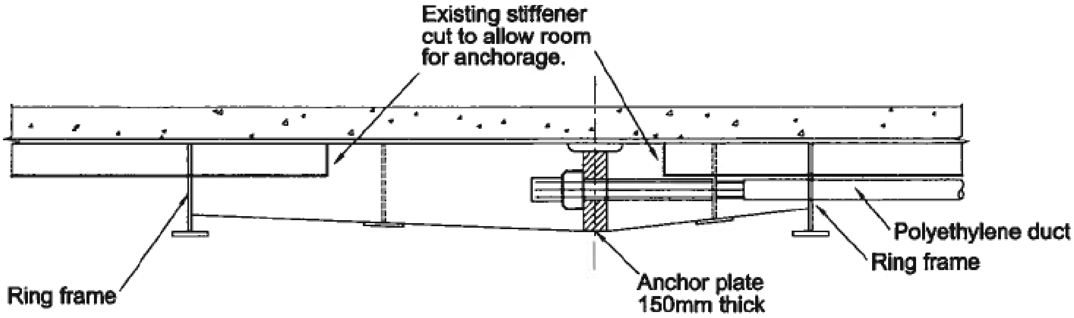


Figure 7.8 Anchorage of the cables to the 150mm plate [3].

7.1.5 Conwy Bridge—diagonal brackets

Conwy Bridge is carrying A55 in North Wales see Figure 7.9 and has a deck of 6.7 m carriageway and 1.8 m footway, see Figure 7.10 Inspection of the bridge showed that cross-members supporting the deck plates and the footway were overstressed by local wheel load and accidental wheel load respectively [3].

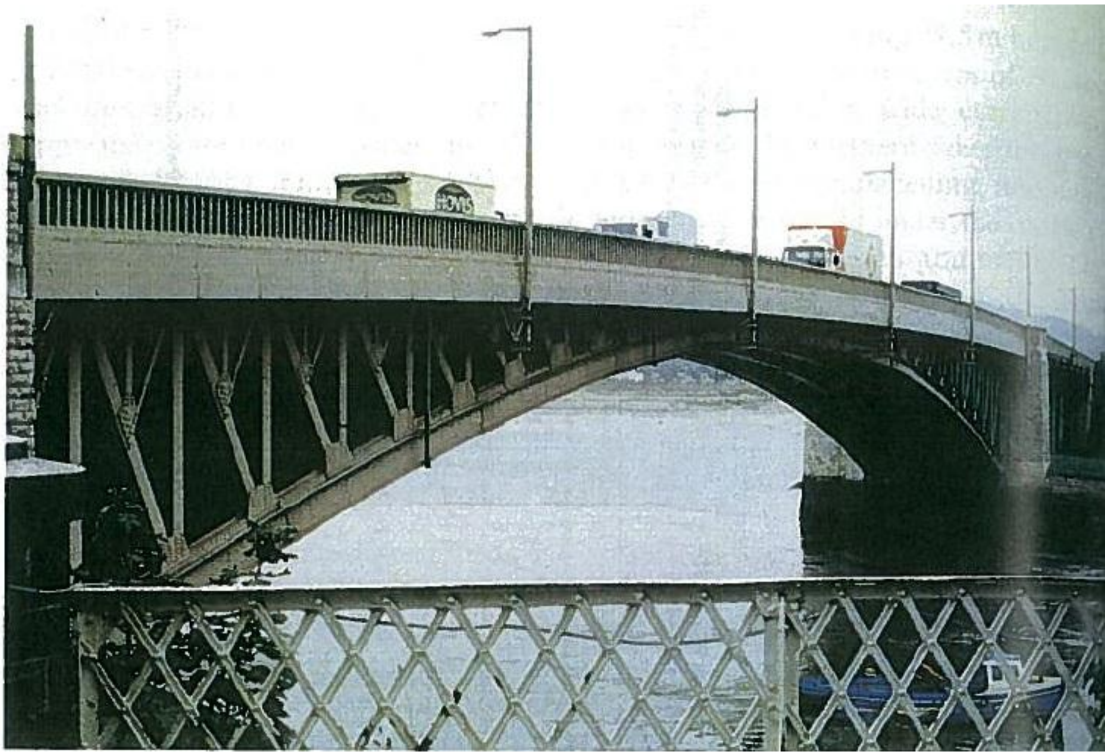


Figure 7.9 A view of Conwy Bridge [3].

Strengthening work limited: A new parapet was installed to the edge of the footway (between the footway and carriageway). Concrete deck next to the footway was replaced by steel decking see Figure 7.10 [3].

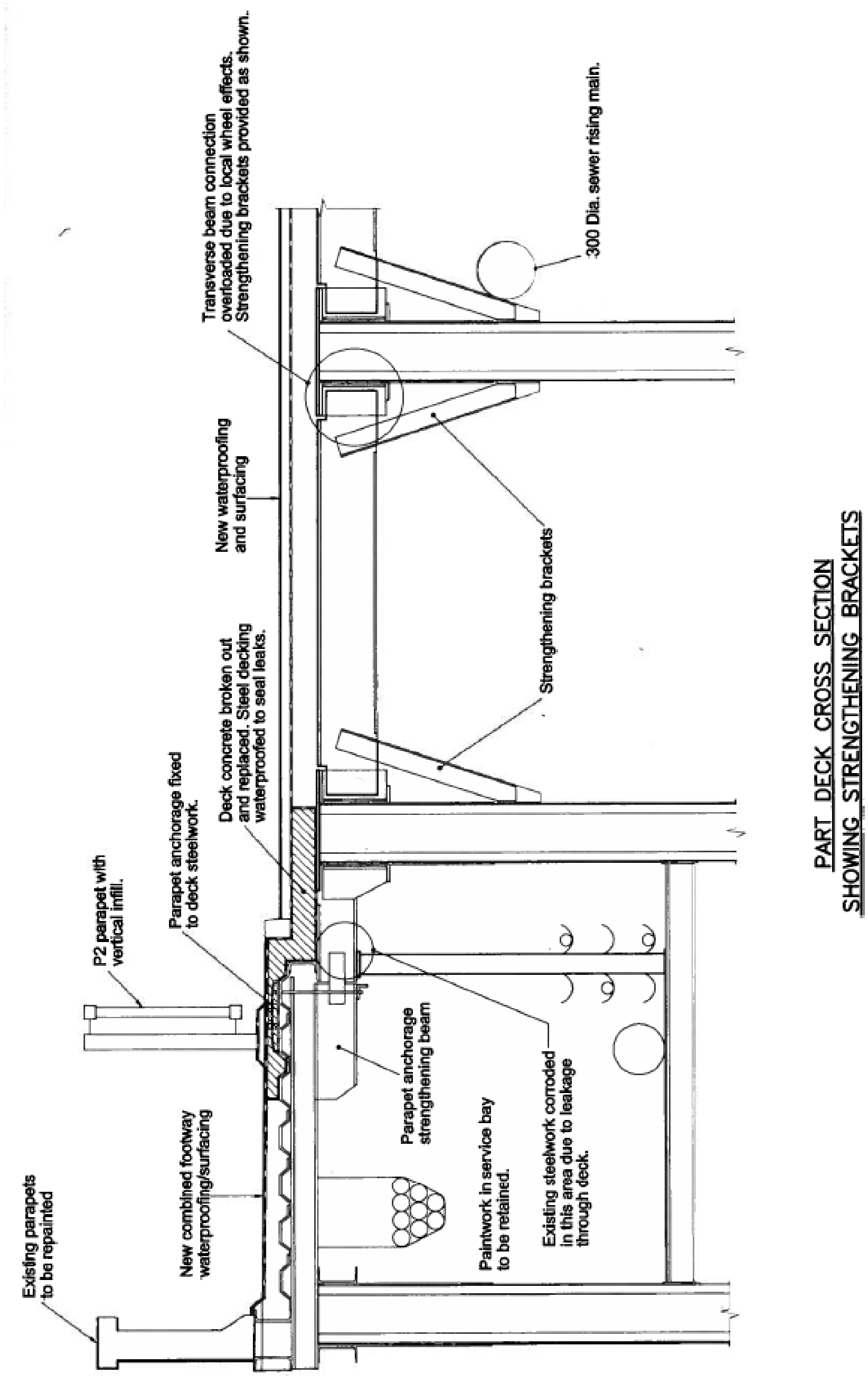


Figure 7.10 Conwy Bridge [3].

Installation of strengthening: Cross members were supported by knee brackets (diagonal struts) see Figure 7.11 [3].

Figure 7.11 Diagonal struts-Conwey Bridge [3].

Table 7.5 below is containing brief information about the operation of the strengthening [3].

Table 7.5 The facts from strengthening operation [3].

Inspection before the operation	The internal lattice structure inspected.
Tests before the operation	Steel grade tested.
How fasten?	The brackets were connected to the structure by HSFG bolts, see figure 7.11. New parapet was jointed to the new brackets by prestressing bars see Figure 7.10.
Traffic	Traffic was limited to 7.5 t weight during installation of the bracket and traffic was diverted for heavier loads.
Tests after the operation	Welding between the stiffener and the lower rib was tested by MPI.
Time	Generally yes, the operation done in estimated time.
Economic aspects	Generally yes, The operation done in estimated cost.

Problems during the operation	The variation in the geometry was more complicated.
Anything went badly?	Modification of the anchorage system of the parapet needed.
Changes needed if do again?	No.

7.1.6 Wave bridge—flange plates

Wave Bridge, Maldon, UK is built in 1910, see Figure 7.12.

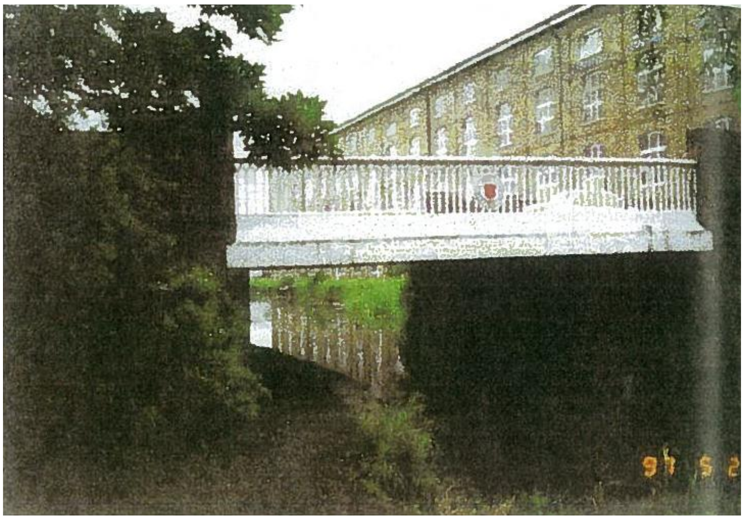


Figure 7.12 A view of the Wave Bridge [3].

This bridge has a deck that carries a single way over a canal. Figure 7.13 shows the deck section that is including riveted steel troughs on abutment of bricks [3].

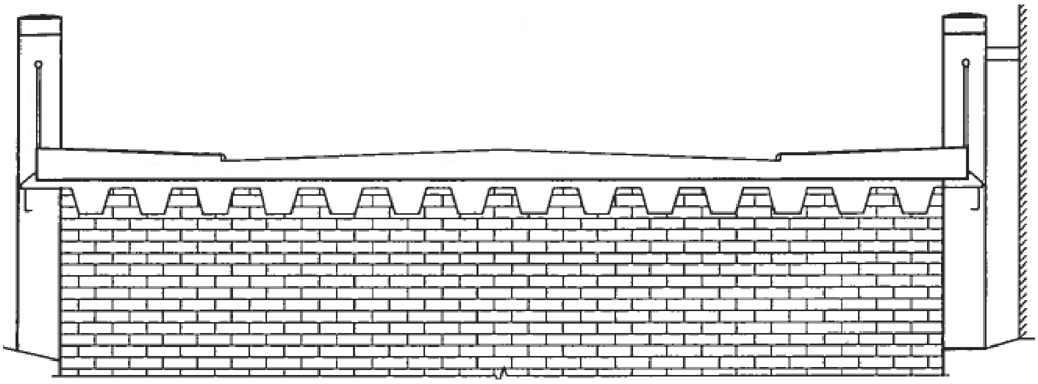


Figure 7.13 Cross-section of the bridge deck [3].

The cross-section of the steel trough is showed in Figure 7.14 [3].

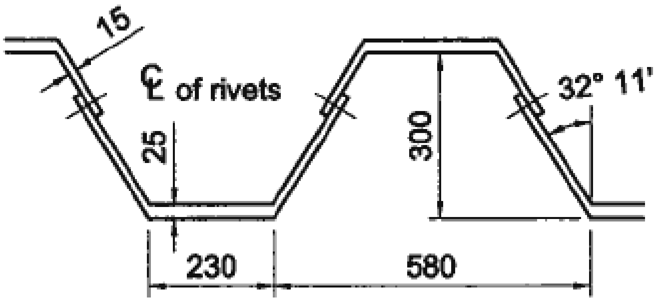


Figure 7.14 Section of the steel trough [3].

The assessment of the bridge showed that the bridge could not withstand a 40 t loading. There was a risk for sagging of the troughs in the middle of the span [3].

Strengthening work limited: The limitation of the load coming from heavy vehicles could not be applicable because of there were no other ways for vehicles to take. Another alternative was enlargement of the deck (or replacement of the deck), but this alternative was eliminated due to the vicinity of existing buildings around. Strengthening of the bridge by adding plate or materials to the bottom side of the bridge was limited due to required headroom [3].

Decision of strengthening: After some studies, installation of the steel plate to the bottom flange of the troughs by welding was decided, see Figure 7.15. The paint system also decided to be renewed [3].

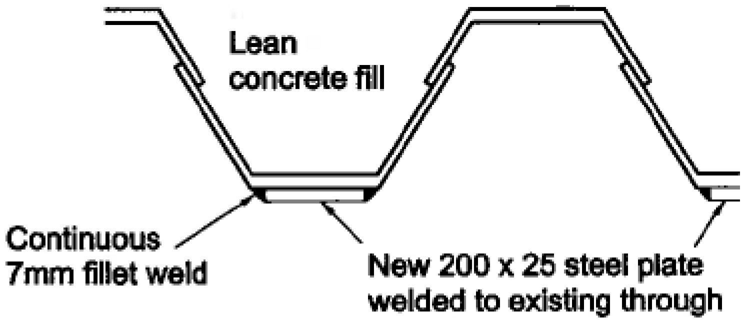


Figure 7.15 Added steel plate showed [3].

Table 7.6 below is containing brief information about the operation of the strengthening [3].

Table 7.6 Information of strengthening operation [3].

Inspection before the operation	Existing steel was inspected. The weldability and the strength of the steel were examined. The paint system was also inspected for lead content (for water under the bridge).
Strengthening works	Due the lack of the access to underside of the bridge and to avoid the disturbance of the water traffic, a working platform was built see Figure 7.16. This platform was built of removal section to enable the pass of the vessels. The steel plates were tack welded to existing steel and then the steel plates were welded continuously.

	Finally the paint system was renewed after blast cleaning.
Traffic	None management of traffic.
Tests after the operation	MPI testing of the welds and the paint system was controlled by an electronic thickness gauge.
Time	Within the cost plan.
Economic aspects	Yes, the operation done within the estimated cost plan.
Problems during the operation?	Large vibrations when the heavy vehicles pass over the bridge.
Anything went badly?	When the platform was erected first time, it was not correctly built. A water surge was happened and the platform was lifted by water and clamped in place.
Changes needed if do again?	The ends of the steel plates could be manufactured tapered or curved. This should help weld operation and also reduce the fatigue risk.

Figure 7.16 Working platform [3].

7.1.7 Honey Lane Bridge—flange channels to steel I-beams

Honey Lane Bridge, UK is located over a channel in Waltham Abbey and is a single-span bridge, see Figure 7.17 [3].

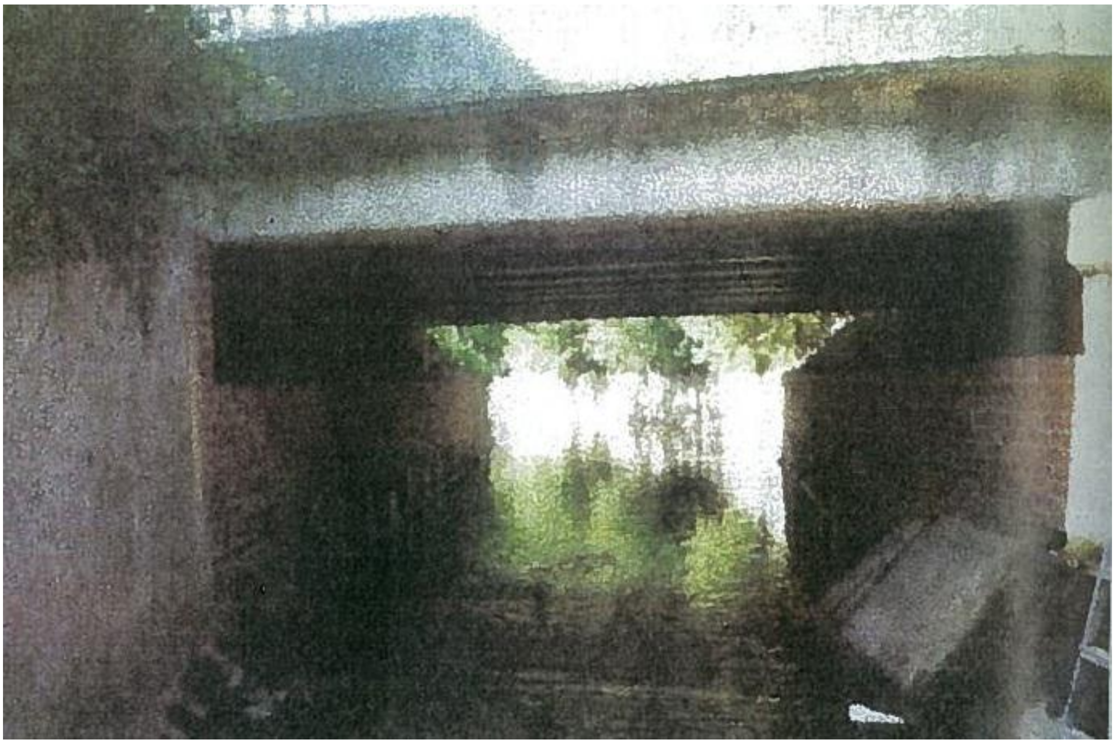


Figure 7.17 Honey Lane Bridge [3].

The assessment of the bridge showed that the bridge could not withstand a 40 t loading. There was a risk for sagging of the troughs in the middle of the span (same as study case 6) [3].

<u>Strengthening work limited</u>: The limitation of the load coming from heavy vehicles could not be applicable because of there were no other ways for vehicles to take. Another alternative was enlargement of the deck (or replacement of the deck), but this alternative was eliminated due to the vicinity of existing buildings around. Strengthening of the bridge by adding plate or materials to the bottom side of the bridge was limited due to required headroom. The Environment Agency required that the lowering of the bridge deck (or soffit level) could be maximum 25mm [3].

<u>Decision of strengthening</u>: After some studies, installation of the steel plate to the bottom flange of the beams by welding was decided [3].

Table 7.7 below is containing brief information about the operation of the strengthening [3].

Table 7.7 Information of strengthening operation [3].

Inspection before the operation	Existing steel was inspected. The weldability and the strength of the steel were examined. The paint system was also inspected for lead content (for water under the bridge).
Strengthening works	The steel plates were tack welded to existing steel and then the steel plates were welded continuously. Finally the paint system was renewed after blast cleaning.

Traffic	None management of traffic.
Tests after the operation	Ultrasonic and MPI tests for welds. The paint system was controlled by an electronic thickness gauge.
Time	Yes, within the cost plan.
Economic aspects	Yes, within the estimated cost plan.
Anything went badly?	Difficulties with welding process (welding trough to the bottom flanges).
Changes needed if do again?	None.

7.1.8 North Bridge—additional beams

North Bridge is located in Colchester, UK [3].

The bridge consisted of the cast iron beams and plates in the side spans that were overstressed. In the middle/ centre span steel beams that had replaced could not resist the full load because of the buckling caused by unrestrained top flange [3].

<u>Installation of strengthening</u>: See table 7.8 under “how fasten?” and “how fit?” and see also Figure 7.18.

<u>Considered alternatives</u>: An alternative was that replace the deck with a 300 mm reinforced concrete slab but it would result in traffic problems [3].

Table 7.8 below is containing brief information about the operation of the strengthening [3].

Table 7.8 Information of strengthening operation [3].

Inspection before the operation	Inspected full principally.
Tests before the operation	Steel grade and weldability properties.
How fasten? How fit?	Universal column (UC) was added to the underside of the side spans and UC supported rectangular hollow sections (RHSs) to carry the deck. To support UC beams stub beams were casted into the abutment and prier walls. In the centre span welding added to reduce the movement of the top flange. See Figure 7.18.
Traffic	Not permitted except Sundays (for loading and unloading).
Tests after the operation	MPIs.
Time	Not in to estimated time (4 weeks more than planned).

Economic aspects	£ 15000 more than planned.
Problems during the operation?	The existing steel work was badly corroded and therefore it required an addition of welding and plating.
Anything went badly?	The dry packing (between the new steel work and the existing soffit).
Changes needed if do again?	Changing/improvement of the dry packing specification.

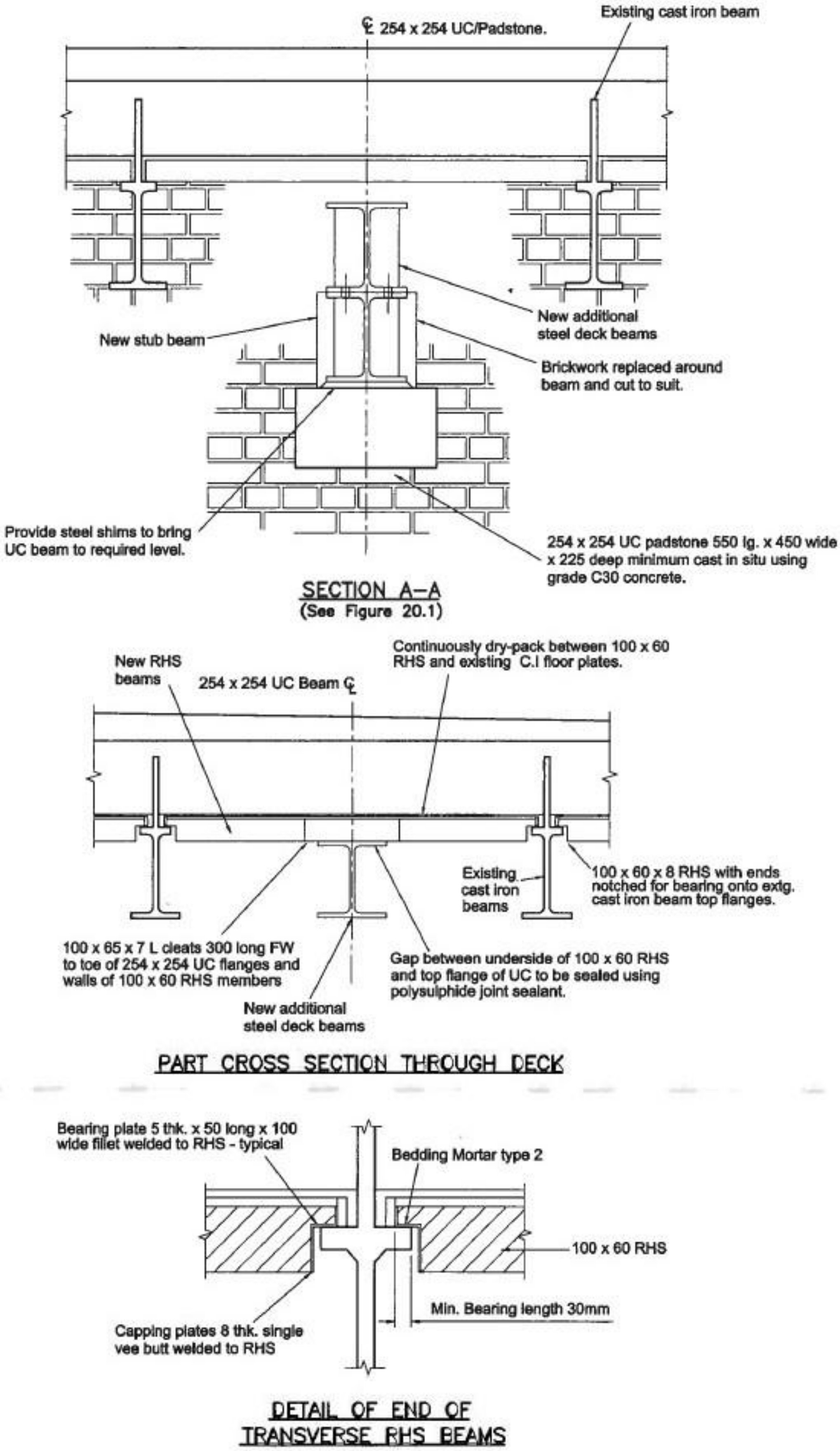


Figure 7.18 Strengthening works of North Bridge [3].

7.1.9 Avonmouth Bridge—prestressing

Avonmouth Bridge is carrying M5 and is located in UK, see Figure 7.19.

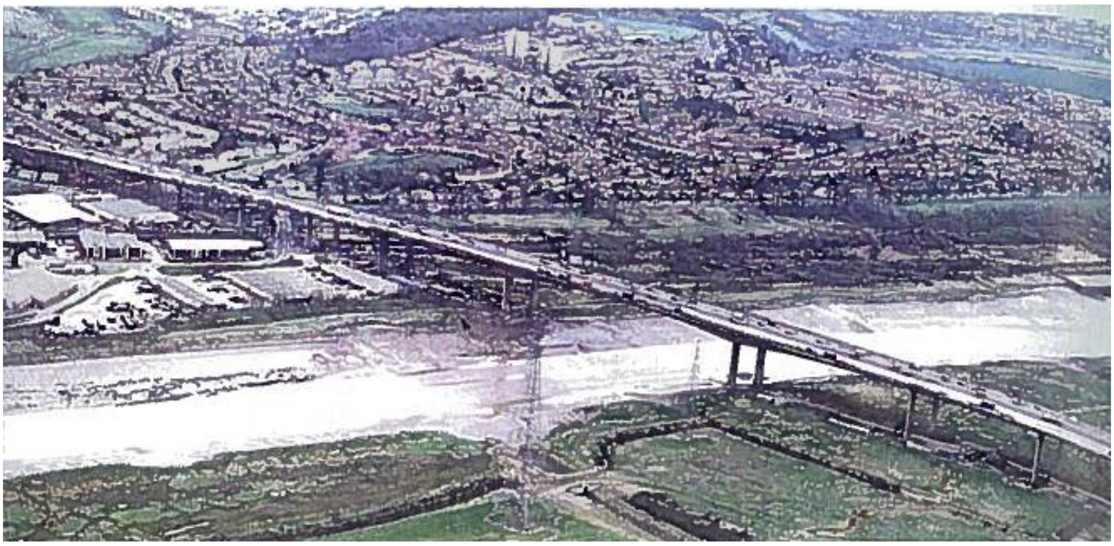


Figure 7.19 A view of Avonmouth Bridge [3].

Figure 7.20 shows the approach and main spans in the bridge structure. On the approach spans the bridge has a 1400 m twin box girder with composite concrete deck and an orthotropic steel deck on the hunched 174 m main span and side spans [3].

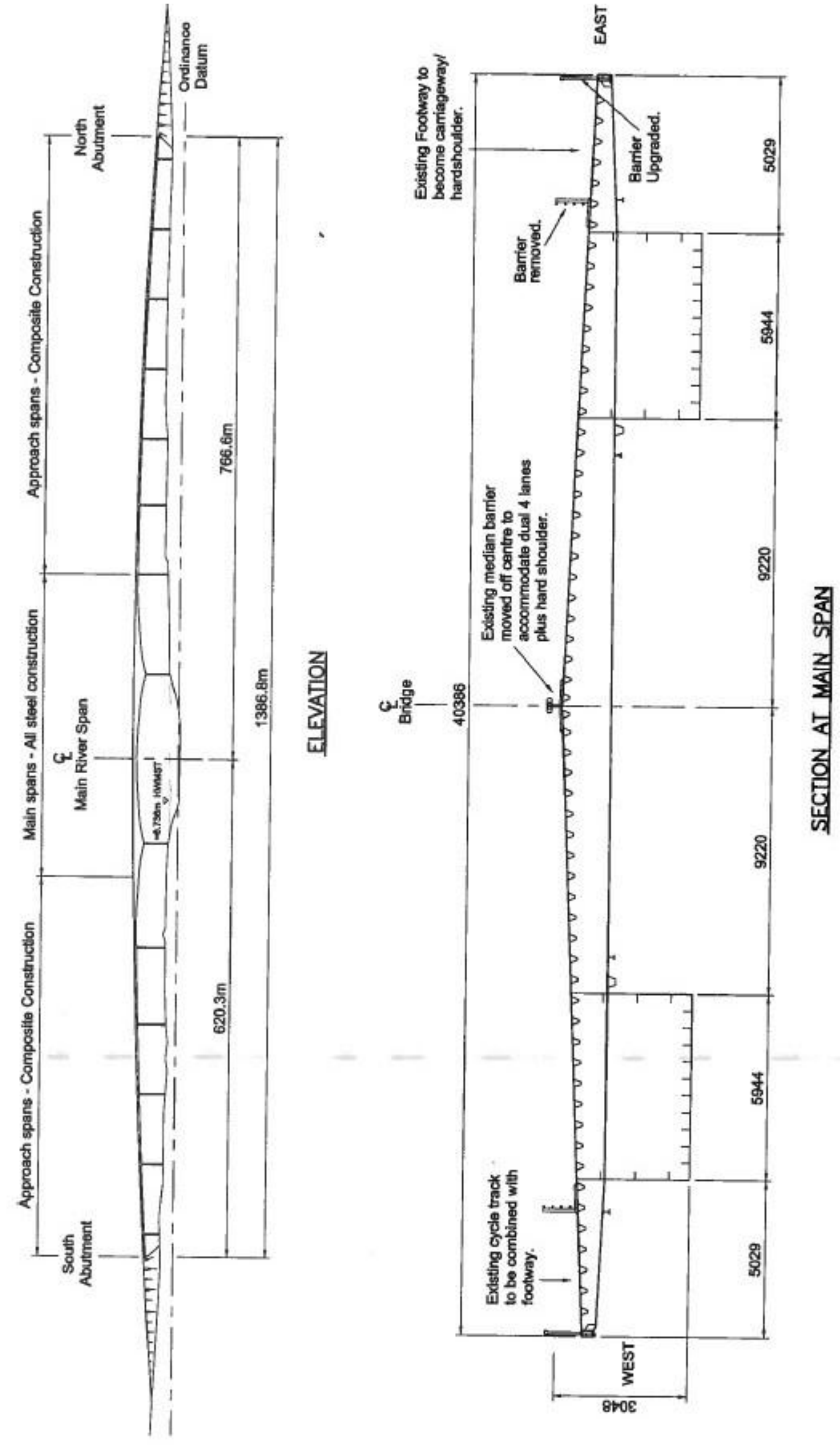


Figure 7.20 Drawing of Avonmouth Bridge [3].

The bridge was designed for a dual three-lane road and hardshoulder, but now the bridge had to be strengthened to carrying a four-lane road and hardshoulder and a cycleway/footway without changing the existing width see Figure 7.20 [3].

Distortional bending stresses, tension and compression flange stresses and web shear stresses within the box girders were above the limits due to the additional loads. Problem arose with the cross-girder that connecting the two box girders. The stability and bending resistance of the cross-girders were insufficient due to the additional loads. Another problem was the V-troughs of the orthotropic deck that not managed the shape limitation requirements in which means a reduction of the effective section [3].

Strengthening work limited: Footway loading was reduced due to British codes. The hardshoulder had not to be used for an event (e.g., as running lane).

Installation of strengthening: Figure 7.21 and 22 show how Macalloy bars were installed in the approach spans.

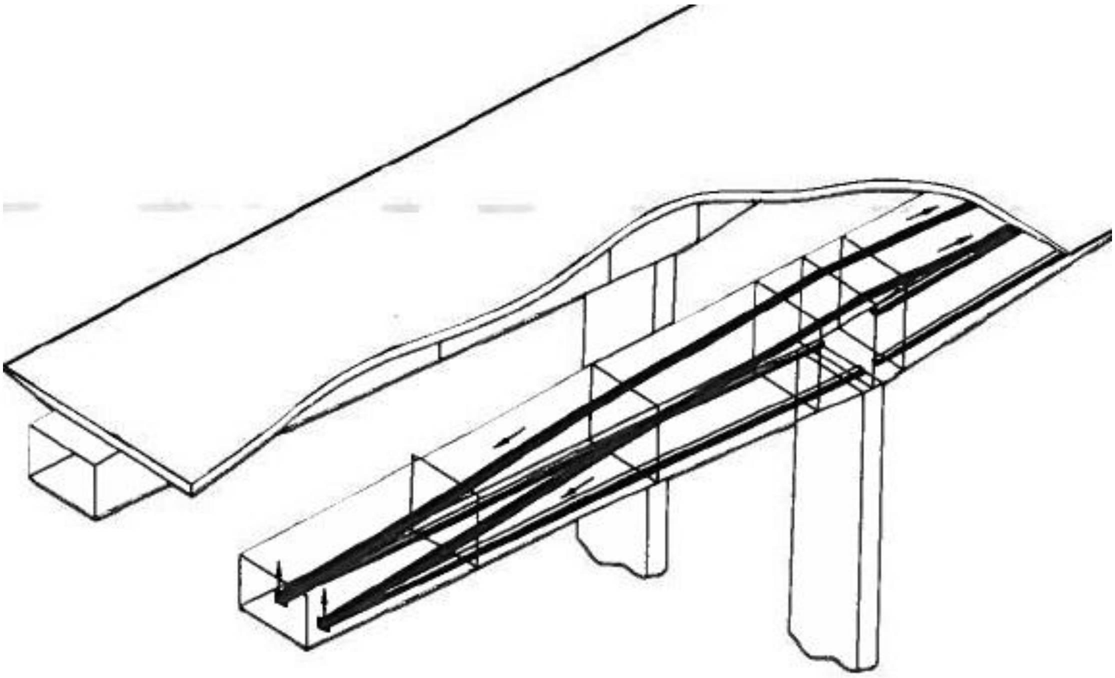


Figure 7.21 A view of Macalloy bars in the approach span [3].

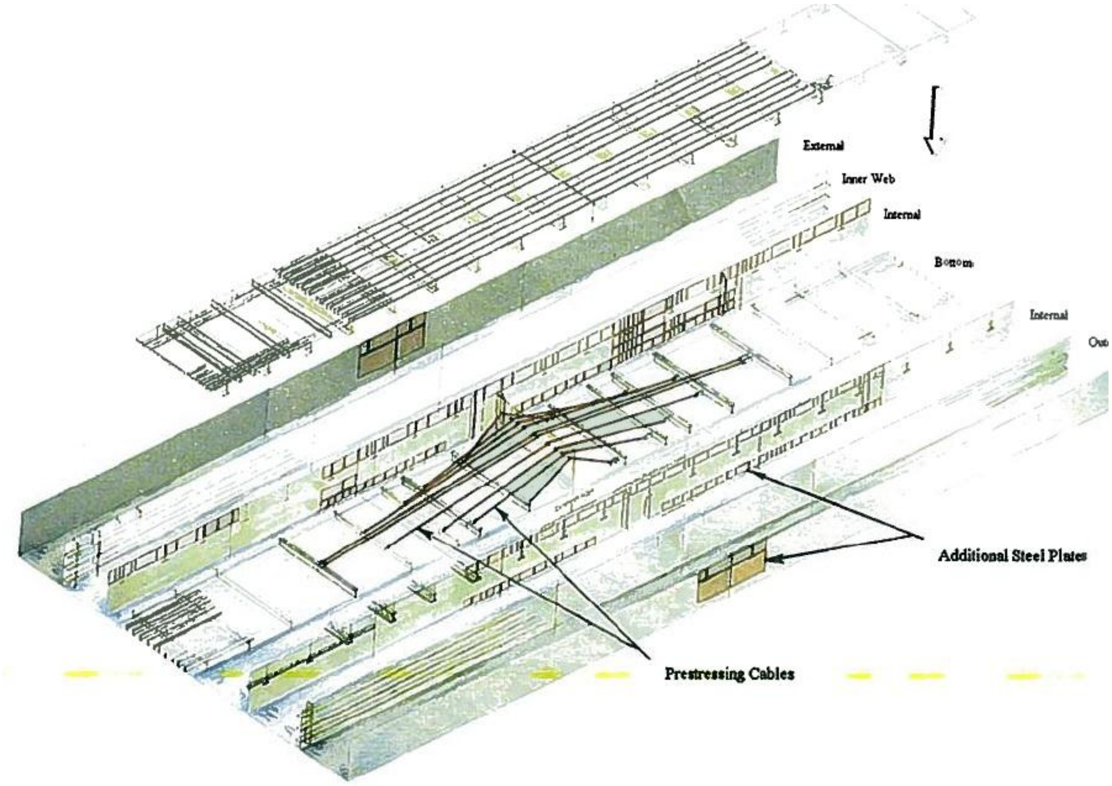


Figure 7.22 Macalloy bars in the bridge [3].

<u>Considered alternative</u>: Additional plate to the box girders was considered but this method was eliminated due to safety issues (during installation, future inspection etc.) [3].

Table 7.9 below is containing brief information about the operation of the strengthening [3].

Table 7.9 Information of strengthening operation [3].

Inspection before the operation	Detailed fatigue tests on the critical details. Straightness and flatness of the existing plate and stiffeners were inspected.
Tests before the operation	Strain gauge checks under abnormal load.
Traffic	Three lanes in each direction kept opened to traffic. The load from traffic controlled under welding and an alarm system was installed when a load exceed the assumed load during welding works.
Tests after the operation	Strain gauges for the Macalloy bars and prestrssing cables.
Time	Not exactly (unforeseen condition of the bridge caused the delays of the plan).
Economic aspects	Generally yes, within the estimated cost plan (some modification of the costs have been done)

Problems during the operation?	After the paint removed some fatigue cracks of the welds detected.
Anything went badly?	No.
Changes needed if do again?	Not major changes (target price contract can be done more accurately).

7.1.10 New Road Overbridge—heat straightening

New Road Overbridge, UK is continuous structure and has three priers, see Figure 7.23. This bridge is carrying a road over the M5 [3].

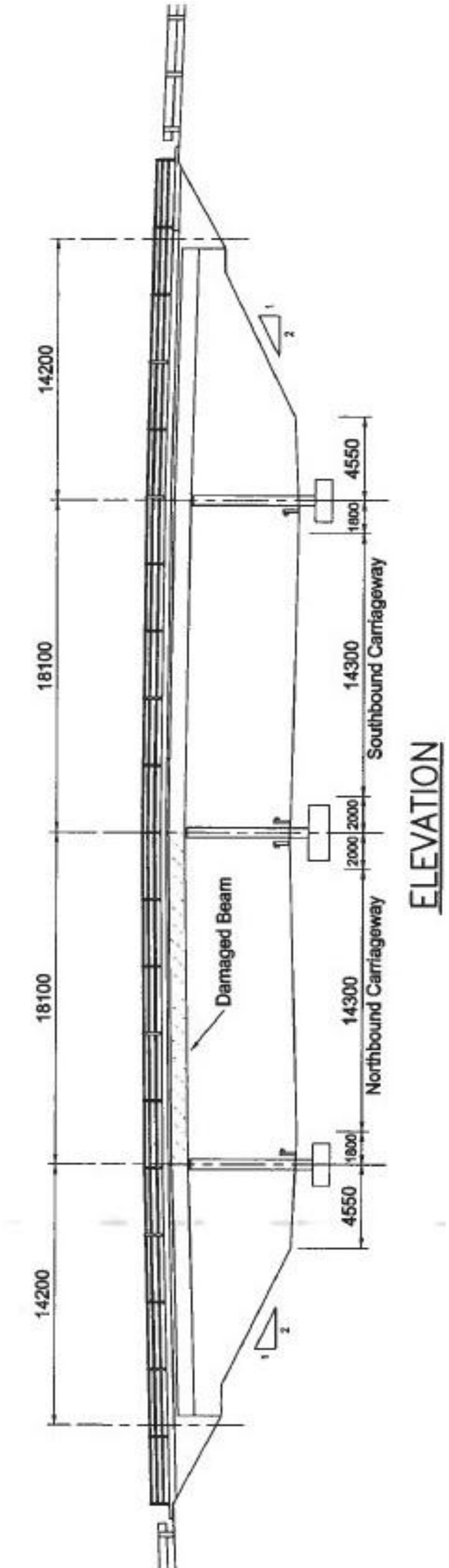


Figure 7.23 New Road Overbridge in UK [3].

The bridge has four universal beams in pair and spliced on each internal support so it creates a continuous system. The beams act compositely with the concrete deck [3].

An accident had happened on the southernmost beam and damaged bottom flanges over the whole three lanes. The maximum displacement of the bottom flange was occurred over lane 1 and was 500 mm; see Figure 7.24 [3].

Figure 7.24 Straightening work of damaged beam [3].

Strengthening work limited: Access to the bridge allowed under night and the quickest repair is required [3].

Installation of strengthening: Heat straightening operation and addition of two bracings and two web stiffeners [3].

Considered alternatives: Cut-off and replacement of the damaged beam were considered but both of them were eliminated due to economic aspects. Both methods were more expensive than the heat straightening [3].

Table 7.10 below is containing brief information about the operation of the strengthening [3].

Table 7.10 Information of strengthening operation [3].

Inspection before the operation	The damaged beam was tested under night. MPI tests had been done for shear studs in the concrete deck and a damaged beam.
How heated?	"Vee" heating used in the bottom flange. Line heating used to the web where the yield zone existed on the top of the web. Vertical strip also heated to match with the bottom flange heating. The heating temperature was between 550 and 600°C due to under 650°C the steel properties not changed. The heating controlled by naked eyes (colour of the steel below the oxy-acetylene flame).
Traffic	Contraflow was planned to install for three weeks on

	M5 but after two weeks it was removed. The lane (1) over the damaged beam was closed to traffic since the accident happened.
Tests after the operation	MPI testing on the whole beam. Chemical analysis, microstructural examination, grain flow determination, hardness, Charpy impact test and tensile test were done of two middle regions of bottom flange and one middle region of the web.
Time	The work was planned to finish in three weeks but delayed two weeks.
Economic aspects	Within the estimated cost.
Problems during the operation?	The damaged beam was stiffer than estimated.
Anything went badly?	No.
Changes needed if do again?	More analysis in the beginning of the strengthening programme.

7.1.11 Erskine Bridge—collision damage

Erskine Bridge is placed 9 miles of Glasgow, Scotland over the River Clyde see Figure 7.25. The bridge is a cable stayed steel box girder bridge with 15 continuous spans and the main span has distance of 305 m. Total length of the bridge is 1321 m [3].

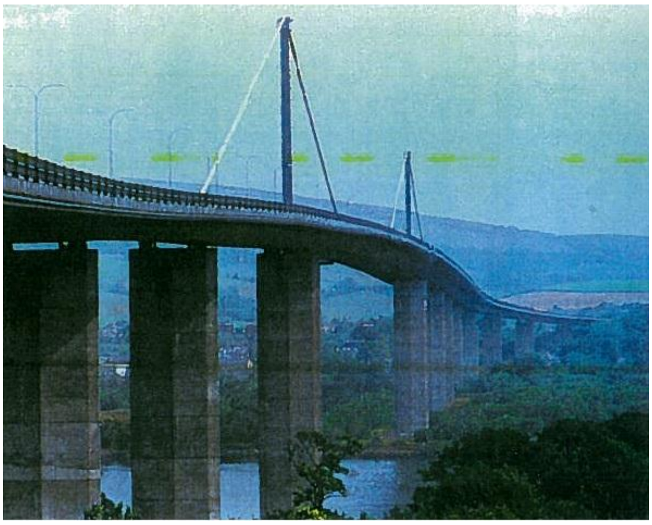


Figure 7.25 Erskine Bridge [3].

In August 1996, a collision was happened between the bridge and an oil rig when the oil rig passed the river. The collision resulted in damage in the bottom flange, see Figure 7.26 [3].

Figure 7.26 Damage caused by a collision [3].

Strengthening work limited: Only damage was strengthened [3].

Installation of strengthening: Bridge was strengthened temporarily to re-open the bridge to three ton vehicles and to stop crack propagation holes were drilled at the ends of the cracks [3].

The damaged flange was permanent strengthened by doubler plates. To fill the voids between the flange and doubler plates polysulphide sealant was used [3].

Two transverse stiffeners were installed to each side of the damaged area [3].

Table 7.11 below is containing brief information about the operation of the strengthening [3].

Table 7.11 The facts from strengthening operation [3].

Inspection before the operation	Internal and external inspections had been done to determine the damage and the size of distortions.
Tests before the operation	MPI of damaged regions.
How fasten?	High friction grip bolts used.
Traffic	One carriage was opened during the operation.
Tests after the operation	Draw-wire displacement sensors were installed see Figure 7.27. Strain gauges were installed. MPI tests of welds.
Time	Within the estimated time (30 weeks).
Problems during the operation?	To remove the lead-based paint by Grit blasting was eliminated due to the risk of causing damage on the prestresses bars. Solvent used instead.

Anything went badly?	No.
Changes needed if do again?	No.

Figure 7.27 Sensor to measure displacement [3].

7.2 Case studies 2

7.2.1 Avesta/Krylbo

A railway bridge is located over Dal River in Avesta/Krylbo and is a type of beam and slab bridge. The railway bridge was inspected and some damages were detected see Table 7.12 [19].

Table 7.12 Detected damages under inspection and their position [19].

Detected Damages	**Position**
Outside: Small flaking, graffiti and damage caused by stones (without rust). Inside: A roost, graffiti and broken lattice.	Abutment: South
Inside: Flaking of painting	Abutment: North

In general the painting system was in good condition and no requirement of a maintenance or painting (until 2025) [19].

Some measures:

- Avoiding access to the abutment of unauthorized
- Cleaning graffiti and roosts.

Figures below shows damage pictures taken from the bridge.

Figure 7.28 S and N abutments, on the right side graffiti can be seen and on the left side damage from stone can be seen [19].

Figure 7.29 Flaking of painting, S abutment [19].

Figure 7.30 Graffiti and a roost inside of the bridge [19].

Figure 7.31 On the left side flaking of painting next to a notch in S abutment and on the right side flaking of bearing painting in N abutment [19].

Figure 7.32 On the left side a blowhole without rust in S and N abutment and on the right side a rust under painting [19].

7.2.2 Bengtsfors

A roadway bridge with one moveable span over Dalslands Channel in Bengtsfors was inspected [19]. Table 7.13 shows detected damages and their positions under inspection.

Table 7.13 Detected damages under inspection and their position [19].

Detected damage	Position
Damage from stone throw (from traffic or persons)	Main beams
Rust on bottom flange	Abutment: East
Damage from stone throw	Abutment: East
Rust	Top flanges against concrete
Rust	Splice

Figures below shows damage pictures taken from the bridge.

Figure 7.33 Damage from stone throw and rust on bottom flange, East abutment [19].

Figure 7.34 Rust on the main beam [19].

Figure 7.35 On the left side rust detected on top flange against concrete and on the right side rust occurred on splice [19].

7.2.3 Iggesund

A railway bridge (type of beam and slab bridge) located next to motorway E4 in Iggesund was inspected. The vicinity to motorway resulted in salt exposing on the bridge [19].

Table 7.14 shows detected damages and their positions under inspection.

Table 7.14 Detected damages under inspection and their position [19].

Detected damage	**Position**
Rust, graffiti and stone throw	Abutment: North
Small damage on the stiffener	Abutment: South
Rust	Top flange next to concrete

Critical areas with the damages should be repaired before 2020 [19].

Some measures:

- Cleaning the critical surfaces.
- Avoiding access for unauthorized.
- Addition of noise barrier protects the bride from salt from the motorway.

Figures below shows damage pictures taken from the bridge.

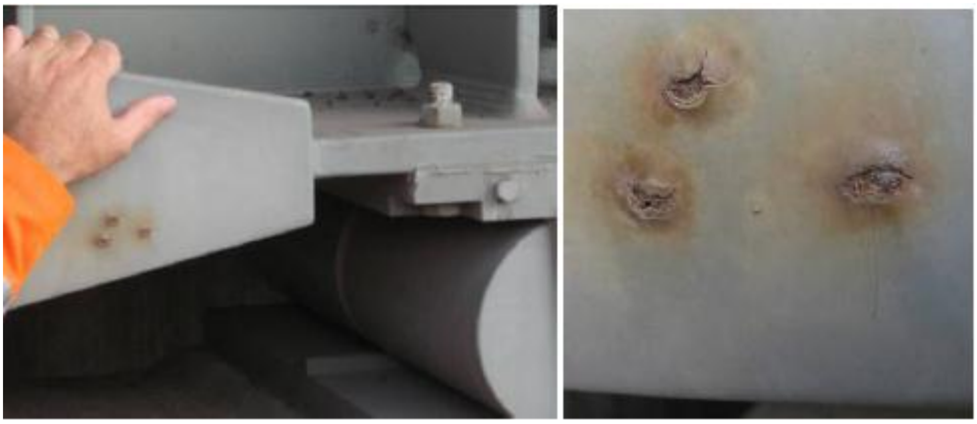


Figure 7.36 Rust in (N) abutment [19].

Figure 7.37 On the left side rust created on a notch and on the right side paint of the bearing loosened (in N abutment) [19].

Figure 7.38 Graffiti, stone throw and rust [19].

Figure 7.39 Detected damage on stiffener [19].

Figure 7.40 Rust created between the top flange and concrete [19].

Figure 7.41 Rust created on cross-beam [19].

7.2.4 Kuserud

A roadway ridge over Krok River alone the motorway E45 was inspected. Detected damages and their positions can be seen in Table 7.15.

Table 7.15 Detected damages under inspection and their position [19].

Detected damage	**Position**
Stone throw	Main beams
Flaking	Main vertical welds of (W) and (E) main beams
A roost and rust	Bottom flange

The paint system was acceptable level in general but maintenance required before 2016 [19].

Figures below shows damage pictures taken from the bridge.

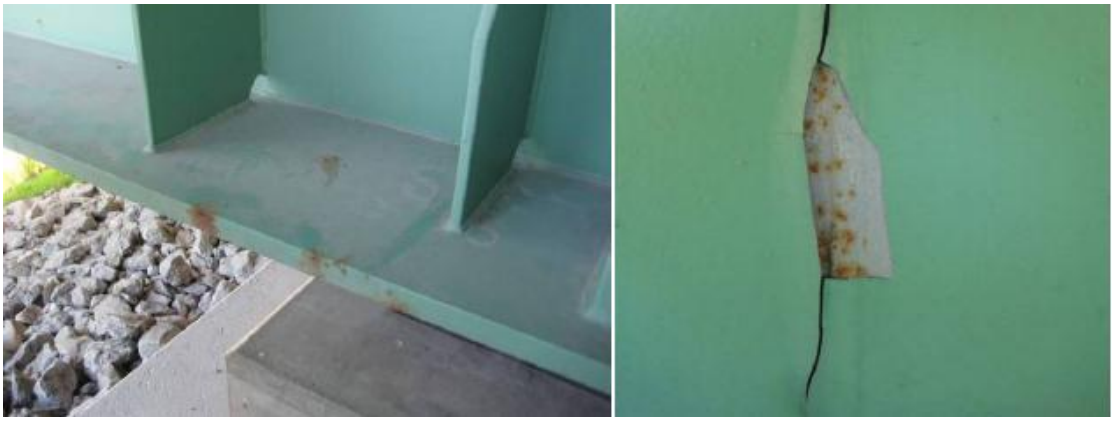


Figure 7.42 On the left side rust created on bottom flange and on the right side cracked paint, paint flaking and rust in main beam can be seen [19].

Figure 7.43 A roost and rust in bottom flanges [19].

Figure 7.44 Paint flaking and rust created on main welds [19].

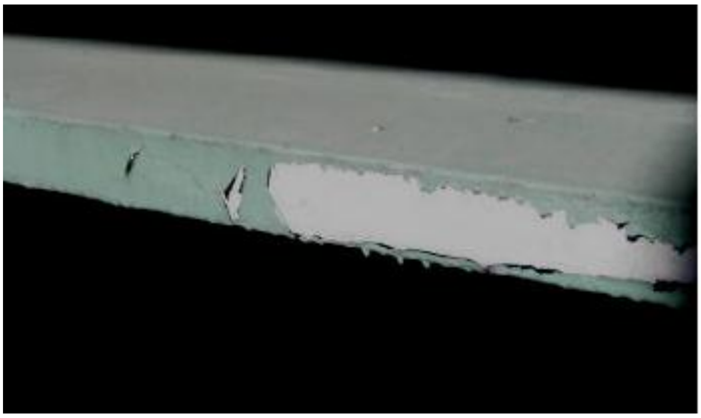


Figure 7.45 Paint flaking on bottom flange [19].

7.2.5 Lerbo

A roadway was inspected. Graffiti, damage from stone throw and rust damages in bottom flange were detected [19].

<u>Recommended measures [19]:</u>

- Maintenance of damaged bottom flange.
- Re-painting of whole structure after 2025.

Figures below shows damage pictures taken from the bridge.

Figure 7.46 On the left side graffiti and stones can be seen and on the right side rust blisters can be seen [19].

Figure 7.47 On the left side rust damage occurred and on the right side rust blisters occurred [19].

Figure 7.48 Rust damages on bottom flange [19].

7.2.6 Nordre älv

Arch Railway Bridge over Nordre River near to Ytterby was inspected [19]. Detected damages and their positions can be seen in Table 7.16.

Table 7.16 Detected damages under inspection and their position [19].

Detected damage	Position
Colour change	---
A few small damages (stone throw from train)	---
Graffiti caused colour damage and rust	Abutment
Corrosion	Some places in edge of the arch

In general the surface of the bridge is in good condition [19].

<u>Recommended measure [19]</u>:

- Improvement of the damages.
- Paint system before 2020.

Figures below shows damage pictures taken from the bridge.

Figure 7.49 Bottom part of the arch was re-constructed and stainless steel used [19].

Figure 7.50 Rust created [19].

Figure 7.51 Dirty components causing corrosion [19].

Figure 7.52 On the left side a damage caused by stone throw by trains can be seen and on the right side damages caused by climbing can be seen [19].

Figure 7.53 On the left side damage caused (have to be repaired) by humans and on the left side fouling can be seen [19].

7.2.7 Smeberg

A roadway bridge over Långevall, River carrying national highway 164 was inspected. Some damages from stone throw were detected in the web of main beam. Deep-seated rusts had been found on the flanges of main beams. The reason to the deep-seated rusts could be stone throw and defect in the beginning of the erection e.g. damages from transport [19].

Two suggestions could be considerable in this case, either re-painting of the whole structure or local improvement of flanges where the deep-seated rusts occurred, before 2015 [19].

Figures below shows damage pictures taken from the bridge.

Figure 7.54 Deep-seated rusts on the flanges [19].

Figure 7.55 On the left side deep-seated rust can be seen and on the right side stone throw damages can be seen [19].

8 Management of bridges in Sweden

Transport and communication network in Sweden consists of 13642 km railway and 98400 km roadway [22]. When terrain is not suitable for roadway and railway, bridges have been built to enable to communication and transport.

In Sweden a bridge must have a span of at least 3 meter length to be called a bridge. This definition was valid until 1998. Many pipeline bridges built of steel were not categorized as a bridge because of their span of 2 meter and they were not maintained and inspected due to bridge standards. In 1998 the definition of the bridge was changed and the new bridge definition became a span of 2 meter length. In USA a bridge shall have a span of 6 meter length to be called a bridge compared to Sweden. [20].

The change of the definition caused an increase of number of bridges. Figure 8.1 shows number of roadway bridges managed by former Vägverket (called Trafikverket today) between 1995 and 2003. In 1998 the number bridges increased by 2500 because of the new definition [20].

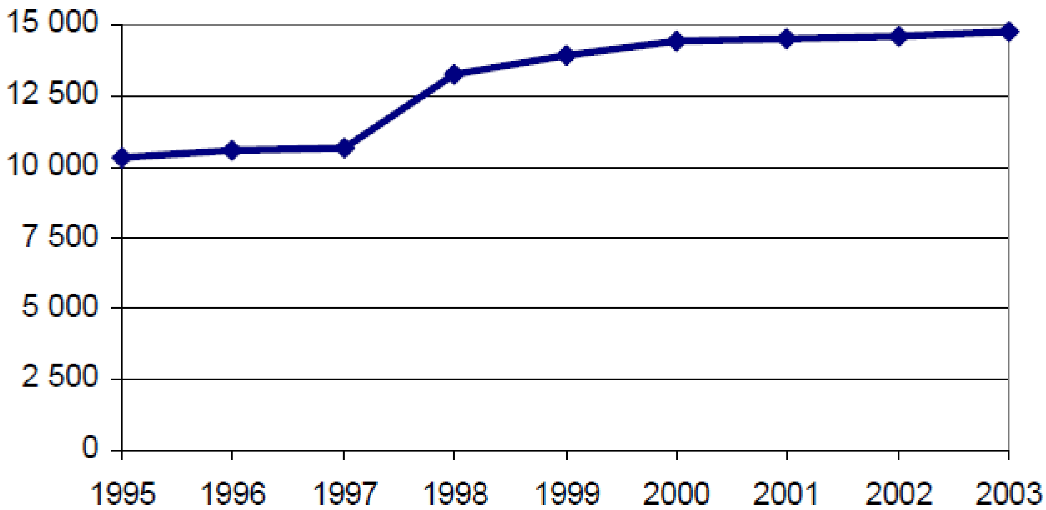


Figure 8.1 Number of roadway bridges managed by old Vägverket (Swedish Road Administration) [20].

Bridges in Sweden are owned and managed by Swedish Road Administration (TRV), communes and private organisations or persons [20]. TRV owns and manages most of the bridges in Sweden and has developed a programme called BaTMan (Bridge and Tunnel management) to manage their bridges, see Section 2.

In BaTMan about 27000 bridges are registered and most of the owned by TRV. Figure 8.2 shows distribution of the bridges regarding to owners. In this Figure distribution of bridges owned by private persons or organisation are not included.

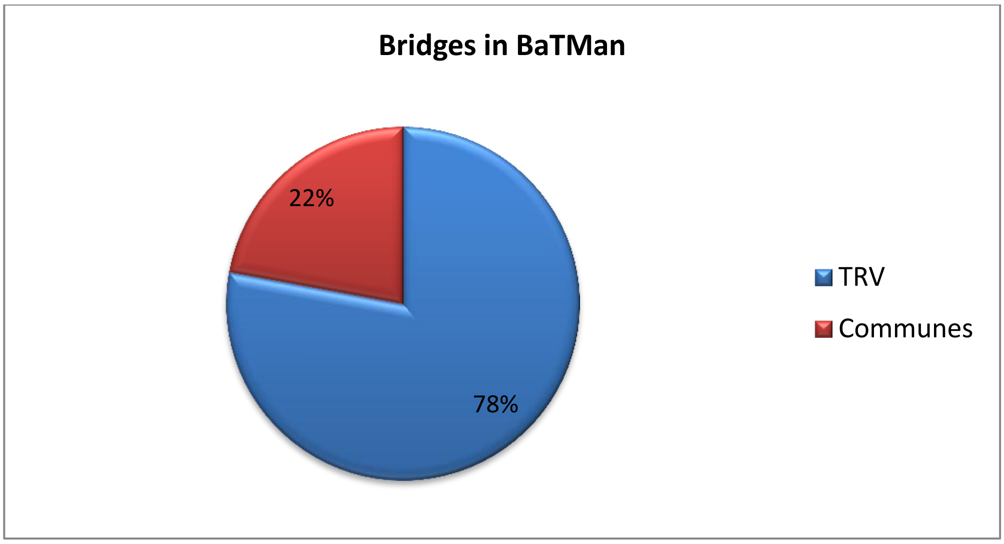


Figure 8.2 Distribution of the bridges in BaTMan with respect to the owner [16].

Figure 8.3 shows distributions of roadway and railway bridges owned by TRV. Roadway bridges are dominating compared to railway bridges.

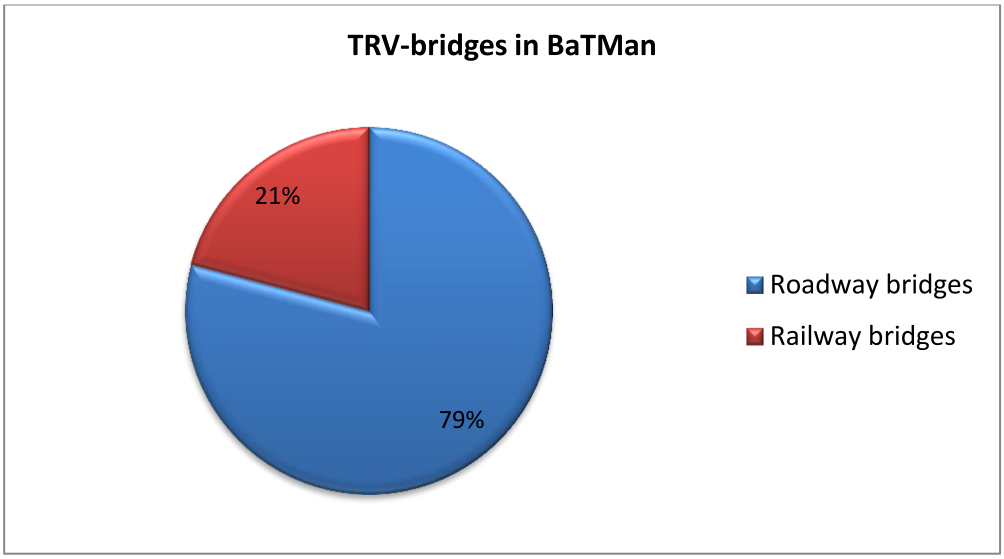


Figure 8.3 Distribution of the road- and railway bridges owned by TRV registered in BaTMan [16].

8.1 Age profile of bridges in Sweden

Figure 8.4 and 8.5 show age profile for railway and roadway bridges owned by TRV, respectively. 53% of TRV´s railway bridges are older than 50 years and are in need of continuous maintenance/inspection and repair.

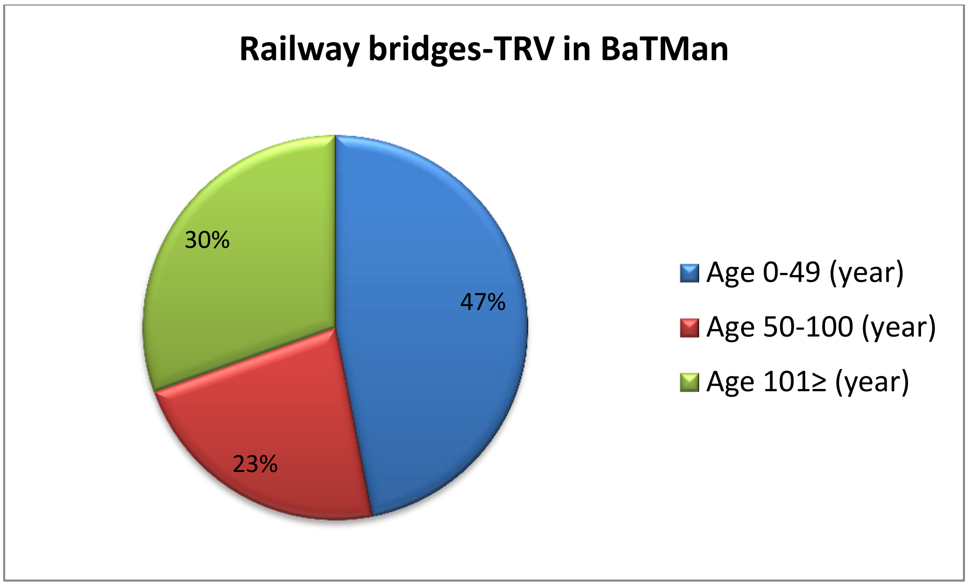


Figure 8.4 Age profile of railway bridges owned by TRV [16].

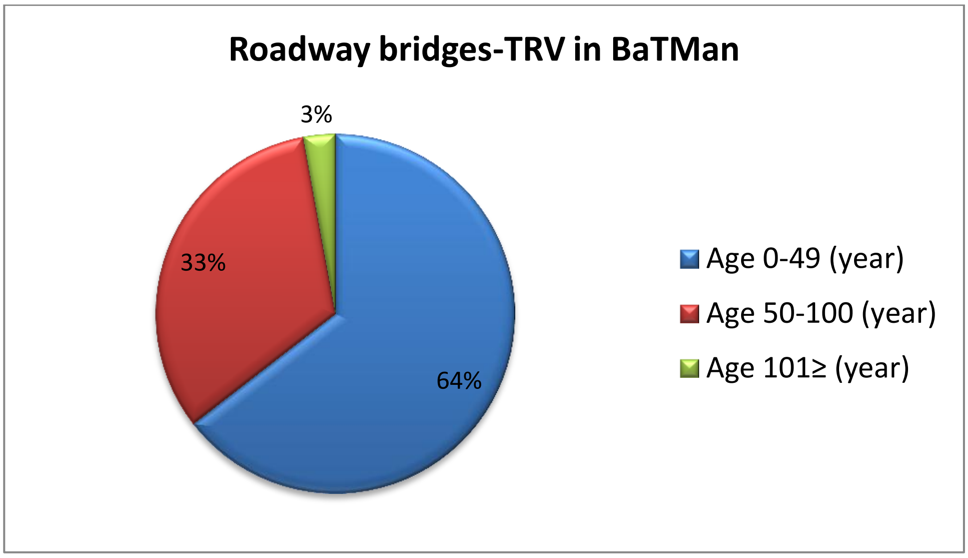


Figure 8.5 Age profile of roadway bridges owned by TRV [16].

Age profile for railway and roadway bridges owned by communes (registered in BaTMan) is presented in Figure 8.6. 32% of the bridges are older than 50 years compared to TRV´s bridges.

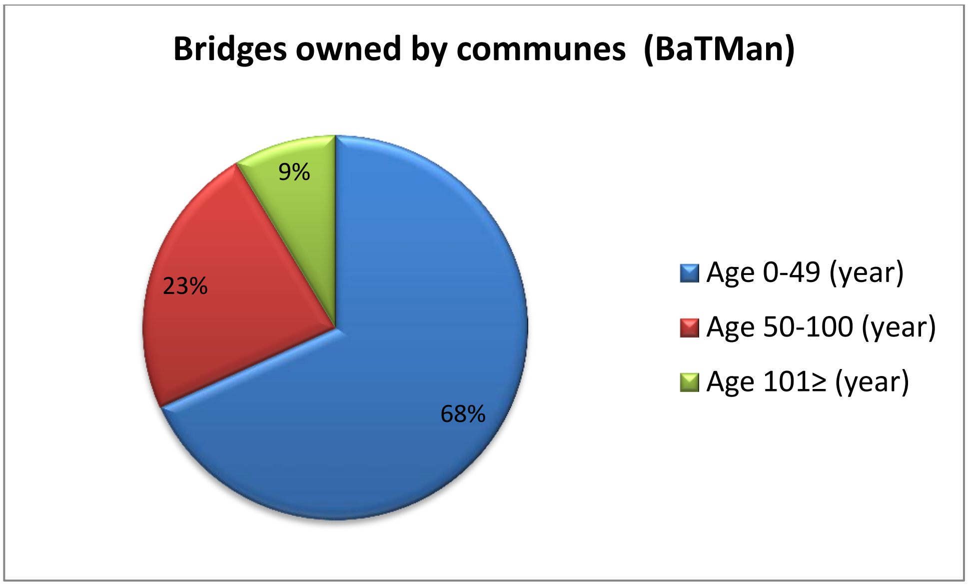


Figure 8.6 Age profile of bridges owned by communes [16].

As can be seen, many bridges are older than 50 years and they often have some type of damage and require continuous maintenance and repair or strengthening.

8.2 Cost perspective of bridges

A typical Life Cycle Cost diagram (LCC) for a bridge is showed in Figure 8.7. Initial investment for a new bridge is rather high. As seen, the costs for maintenance/inspection, repair and demolition or removal of the bridge are also high and have a vital role in LCC. However, the costs for different activities are distributed over many years.

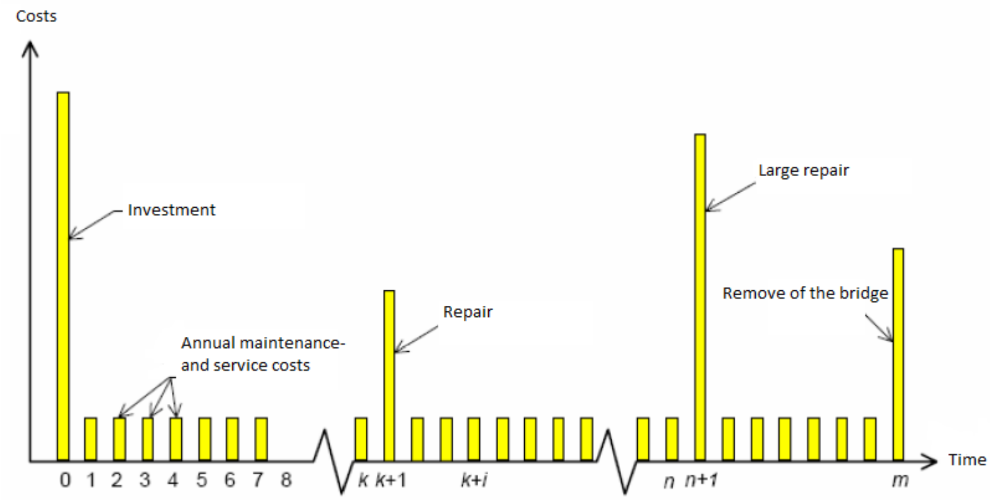


Figure 8.7 Typical LCC analysis for bridges [20].

Figure 8.8 shows investment distribution for different costs (e.g. maintenance, improvement, etc.) and new construction made by TRV between years 1999-2009. TRV spent 8 billion SEK to build new roadway bridges and about 6 billion SEK for improvement, maintenance etc. for roadway bridges to keep them in service. In Appendix 1, investment for different activities made by TRV for railway bridges, by Stockholm commune for both railway and roadway bridges and by Gothenburg commune for both railway and roadway bridges can be found.

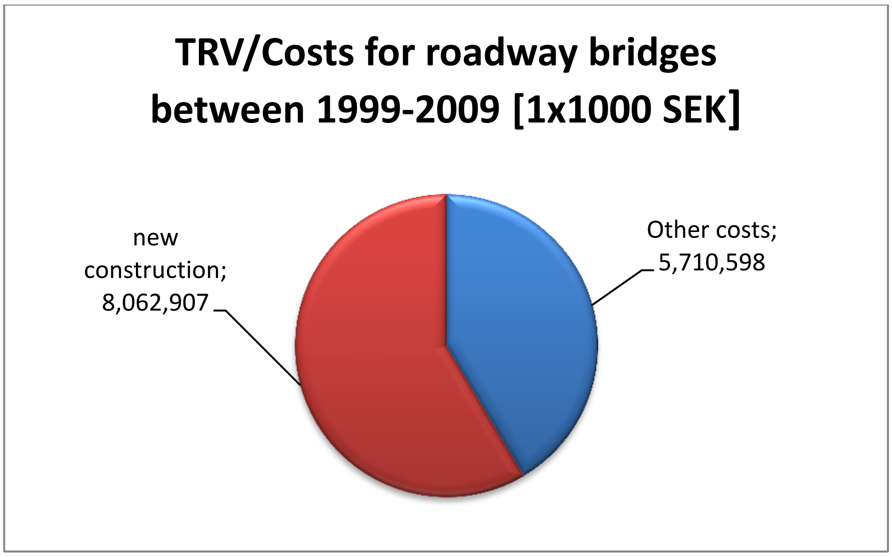


Figure 8.8 Investment distribution made by TRV for roadway bridges [16].

Conclusion that can be drawn is that costs for maintenance and repair show an increasing trend during the past 10 years in Sweden.

An example for cost analysis of bridge deck is shown in Figure 8.9. In first 30 years repair cost is increasing linearly with a mild slope, (in this stage for instance insulation is an issue). After 30 years no repair is applicable and profitable. Therefore re-insulation can be made. This figure is a good example to show how maintenance work is influencing the costs.

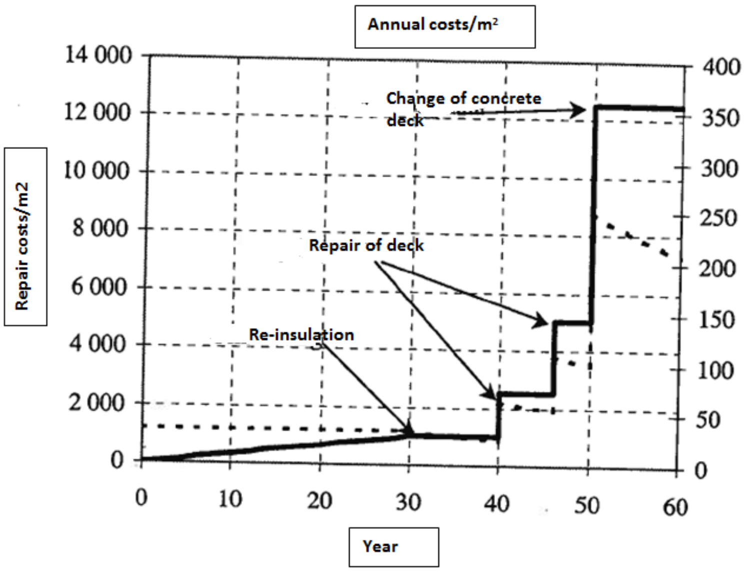


Figure 8.9 Cost developing for bridge deck [21].

8.3 BaTMan and condition of existing bridges in Sweden

BaTMan is a bridge and tunnel management programme and is developed (replaced Safebro) by TRV. BaTMan has about 600 users and about 27000 bridges plus 3000 other constructions registered. These bridges and constructions are owned by TRV, 71 communes and private organisations and persons [16]. In Appendix 2, 71 communes registered in BaTMan can be seen.

Information regarding drawings, inspections, costs for different activities etc. for each bridge can be found in BaTMan [16].

In Sweden, bridges are evaluated based on their condition class. It varies between 0 and 3 which 0 indicates the initial condition and 3 represents the worst.

According to BaTMan the inspector puts evaluation information in the system, e.g. information about damage. Condition classes are as follows.

Table 8.1 Condition class for bridges in Sweden [20] [21].

Condition class	Evaluation made by inspector
0	Failure beyond 10 years
1	Failure within 10 years
2	Failure within 3 years
3	Failure at inspections time

These classes are used as an indicator in bridge management process. For instance, if a damage on construction member is categorized as class 3, this damage and its effect on the member have to be investigated in maximum 3 months and an action should be taken [20].

Table 8.2 shows the Markov chain and gives information about how the condition classes work with respect to time. When a new construction is built, all parts can be classified as class 0. After 10 years 25 per cent of the structure is broken down to class 1 and so on. After 30 years 3,9% of the structure is fallen into class 3. If damage (/damages) is not repaired, results in 11,6% of structure fallen into class 3 after 40 years (10 years after last inspection). The table below is created by factors 0,25, 0,35 and 0,45 and these factors inform that how much of material in per cent is broken down to next deformation class.

Table 8.2 Markov kedja (Markov chain in English) [21].

State class	Part that broken down to next class									
0		**1,000**	0,75	**0,750**	0,75	**0,563**	0,75	**0,422**	0,75	**0,316**
	0,25		0,25		0,25		0,25		0,25	
1				**0,250**	0,65	**0,350**	0,65	**0,368**	0,65	**0,345**
	0,35				0,35		0,35		0,35	
2						**0,088**	0,55	**0,171**	0,55	**0,223**
	0,45						0,45		0,45	
3								**0,039**	1,00	**0,116**
	Time/ year	**0**		**10**		**20**		**30**		**40**

The table above is not working in practice due to made assumptions. Two main assumptions are made to create the table are namely:

- Damages are independent from each other
- Speed of the degradation should be known

Degradation and its rate is not easy to estimate and evaluate, therefore the Markov chain theory does not work in practice, however, it gives theoretical information.

TRV inspect its bridges continuously and register damages in different condition classes in BaTMan. Table 8.4 and 8.5 shows the registered bridges in condition class 3 in BaTMan, for railway bridges and roadway bridges respectively.

Table 8.4 Damages in condition class 3 in Railway bridges owned by TRV [16].

Railway Bridges	**Classification of function (condition class 3)**				
Owner/manager	**Durability**	**Bearing**	**Traffic safety**	**Other**	**Total**
TRV-Middle-	42	41	31	15	129
TRV-East/Stockholm	17	72	14	6	109
TRV-North	1	4	3	5	13
TRV-South	59	55	39	58	211
TRV-West	106	129	124	143	502
Total	225	301	211	227	964

Table 8.5 Damages in condition class 3 in Roadway bridges owned by TRV [16].

Roadway Bridges	**Classification of function (condition class 3)**				
Owner /manager	**Durability**	**Bearing**	**Traffic safety**	**Other**	**Total**
TRV-Middle-	59	54	58	16	187
TRV-North	44	40	45	8	137
TRV-Stockholm	155	94	70	35	354
TRV-South	114	19	24	20	177
TRV-West	343	55	96	66	560
TRV-East	108	68	57	26	259
Total	823	330	350	171	1674

AS seen, 10% of roadway bridges and 23% railway bridges are in condition class 3 and in need for strengthening or repair.

It can be concluded that Region West has many problems and most of damaged bridges are in this region.

TRV has divided its working district in 5 regions for railway bridges and 6 regions for roadway bridges to manage their bridges effectively. These regions are, see Table 8.6:

Table 8.6 Different districts, TRV [16].

TRV-working districts	
TRV-Railway bridges	**TRV-Roadway bridges**
TRV-Middle	TRV-Middle
TRV-East/Stockholm	TRV-East
TRV-North	TRV-Stockholm
TRV-South	TRV-North
TRV-West	TRV-South
	TRV-West

8.4 Interview

A questionnaire was created to get information and evaluation from authorities on new and existing bridges. The aim of the questionnaire was to collect information about bridges and different priorities for repair and strengthening work.

The questionnaire includes three main areas:

- Issues of design and construction of new bridges in densely populated areas
- Issues of maintaining and refurbishing existing bridges in densely populated areas
- Maintenance issues

Table 8.7 shows three names of authorities from different countries that answered questions.

Table 8.7 3 experts that filled the questionnaire.

Name	Job title	Area of responsibility	Country
Valle Janssen	Bridge Engineer	Bridge Maintenance (Railway)	Sweden
K. Elmi Anaraki	Asset manager municipal infrastructure	Bridge Maintenance	Netherland
A. Castillo Linares	Civil Engineer	Bridge Design	Spain

The summary of the results from the returns of the questionnaire is presented in this section.

Regarding the client demands for construction of new bridges, different questions were asked, see Table 8.8. Based on the responses obtained, different authorities have introduced the following demands as their priorities when it comes to construction of new bridges. These demands based on priority are;

1. Initial cost
2. Maintenance costs
3. Short construction time
4. Minimizing traffic disruption
5. Life cycle costs
6. Minimizing the impact on the surrounding environment
7. Measures to minimize the noise coming from traffic flow on the bridge to the surroundings after the construction

Table 8.8 Answers from different experts

	Sweden	*Netherland*	*Spain*
Client demands for the construction of new bridges in densely populated areas	**Ranking**	**Ranking**	**Ranking**
Short construction time	3	5	2
Minimizing the impact on the surrounding environment		2	2
Minimizing traffic disruption on underlying the bridge in case of flyover highway or railway bridges	4	4	3
Measures to minimize the noise coming from traffic flow on the bridge to the surrounding after the construction		3	2
Initial costs	Within LCC	5	5
Maintenance costs	Within LCC	3	4
Life cycle costs (LCC)	5	1	3

Under maintenance issues, rehabilitation activities are evaluated. The results of rehabilitation and strengthening activities can be seen in Table 8.9 and 8.10. The results obtained in this area show that strengthening of decks in steel/concrete composite bridges, replacement of decks, FRP strengthening of concrete, widening of deck and FRP strengthening of steel bridges are the priorities for bridge authorities.

Table 8.9 Different rehabilitation activities evaluated.

	Sweden		*Netherland*		*Spain*	
Rehabilitation activity	**%**	**Priority**	**%**	**Priority**	**%**	**Priority**
Embankment remediation at bridge ends	20	-	-	1	2	3
Underpinning of foundations	1	-	-	2	2	3
Patch repair of damaged brick or masonry	5	-	-	2	2	1
Stitching of masonry (Fondedile type)	0	-	-	2	2	1
Patch repair of corroded metalwork	2	-	-	2	8	3
Patch repair of metalwork due to fatigue	2	-	-	5	8	2
Painting of metalwork	20	-	-	1	8	2
Repair of concrete deck in steel/concrete bridges	0	-	-	4	35	4
Repair of steel decks	0	-	-	5	15	3
Waterproofing	10	-	-	4	8	4
Bearing replacement	1	-	-	2	10	3
Concrete repair	15	-	-	-	-	-
Replacement of deck	20	-	-	-	-	-

Figure 8.10 Different strengthening activities evaluated

	Sweden		*Netherland*		*Spain*	
Strengthening activity	**%**	**Ranking**	**%**	**Ranking**	**%**	**Ranking**
Strengthening of the foundation	1	-	-	3	10	4
Reinforcement of arches	10	-	-	2	2	2
External pre-stressing—concrete bridges	0	-	-	4	2	2
External pre-stressing—metallic bridges	1	-	-	-	2	2
Increasing section—concrete bridges	1	-	-	-	2	2
Increasing section—steel bridges	2	-	-	-	2	2
Strength. deck in steel/conc. Composite bridges	0	-	-	5	5	4
FRP-strengthening—steel	0	-	-	-	10	3
FRP-strengthening—concrete	3	-	-	-	30	3
Replacement of metallic structural members	5	-	-	-	2	2
Additional reinforcement	1	-	-	5	10	3
Additional metallic structural members	3	-	-	4	10	3
Replacement of the deck	70	-	-	5	2	4
Widening of the deck	0		-	1	10	3
Fatigue prevention	3		-	3	1	2

Table 8.12 shows different demands for new strengthening methods from the experts' view. "Application time", "Minimizing traffic disruption", "long term performance" and "Initial and Future maintenance costs" are ranked with high score.

Table 8.12 New strengthening methods for existing bridges evaluated regarding to different activities.

	Sweden	*Netherland*	*Spain*
Demands for new strengthening techniques/methods for existing bridges in densely populated areas	**Ranking**	**Ranking**	**Ranking**
Application time (in connection with lane closure)	-	5	4
Minimizing the environmental impact on the surrounding environment	-	3	2
Minimizing traffic disruption	5	5	2
Minimizing the impact on the surrounding (noise, dust) from construction site activities	2	4	2
Long-term performance	5	3	2
Ease of application	-	1	4
Initial costs	4	2	5
Future maintenance costs	3	3	4

9 Conclusion

The objectives of this master thesis were;

- To map out the need for repair and strengthening for existing bridges
- To give an overview of the different type of remedial works
- To collect information regarding the bridges and priorities with regard to demands for construction of new bridges, rehabilitation activities and demands for new strengthening techniques in densely populated areas.
- To give an overview of bridge management in Sweden

Nearly 70 % of metallic bridges in Europe are older than 50 years and require some type of remedial works. In steel bridges most common problems are fatigue, corrosion and anti-corrosion protection defects.

Trafikverket in Sweden manages their bridges by help of a programme, called BaTMan. In this programme any information about each bridge can be found. Detected damages in the bridges are categorized and registered in BaTMan. 10 % of roadway and 23% railway bridges managed/owned by Trafikverket, are containing damages in class 3 and in need for strengthening and repair.

Regarding to authorities initial and maintenance costs and traffic disruption are important aspects for clients. Strengthening and replacement of decks are dominating in strengthening activities.

9.1 Future studies

Researches have to be made to develop more effective strengthening and repair works.

Studies have to be carried out to develop faster techniques for construction of new bridges and strengthening and repair methods, for instance using prefabricated systems, e.g. using FRP decks.

Knowledge about long-term performance of strengthening and repair techniques has to be increased, e.g. long-term performance of adhesive joints.

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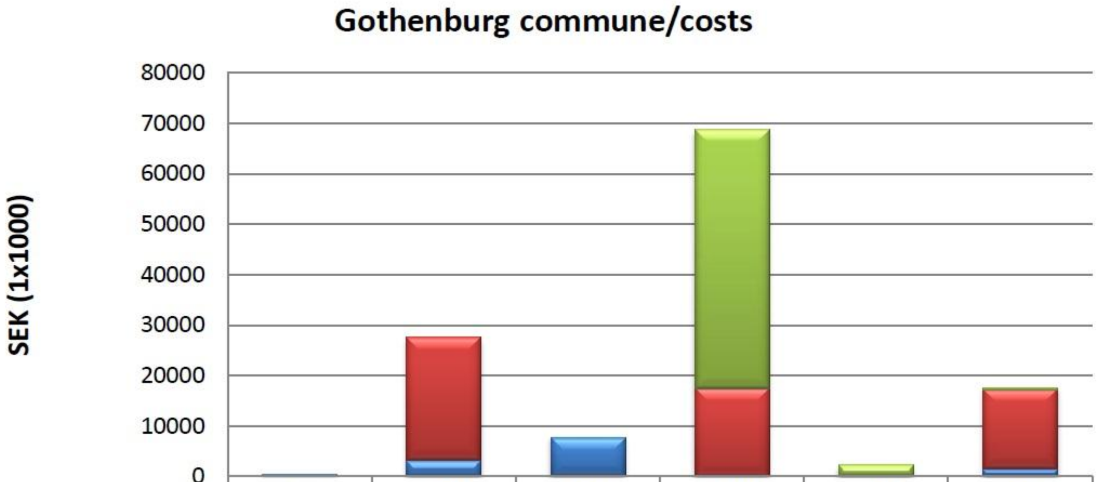
Appendix

Appendix 1: Costs

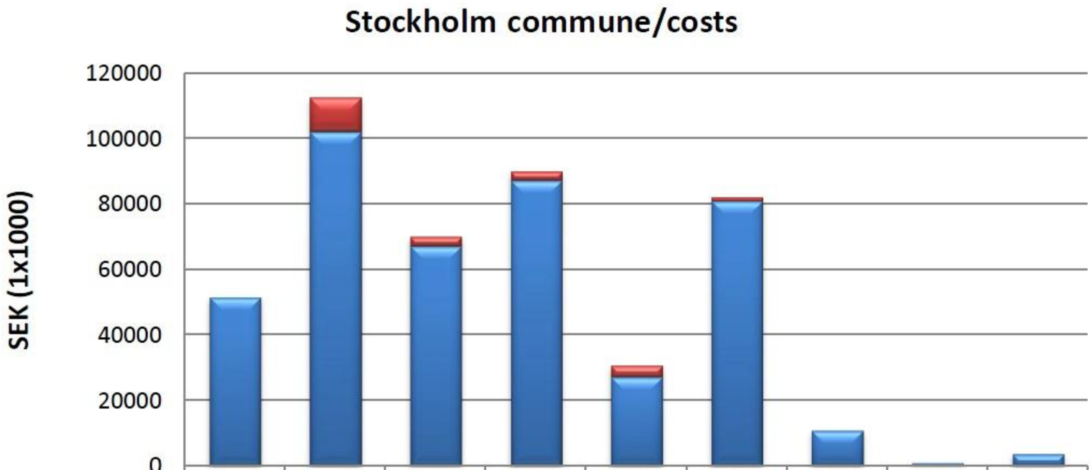
Appendix 2: Communes

Appendix 1, Costs



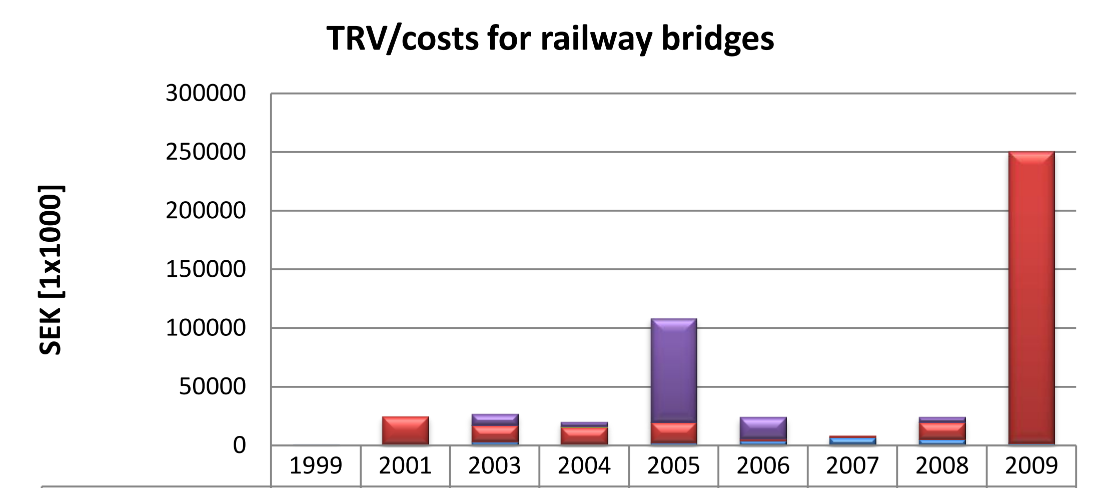
	2000	2002	2003	2004	2005	2006
Improvement	0	0	0	51354	2244	433
New construction	0	24324	0	17370	0	15783
Maintenance	477	3237	7718	0	0	1408

Figure A1.1 Costs made by Gothenburg commune [16].



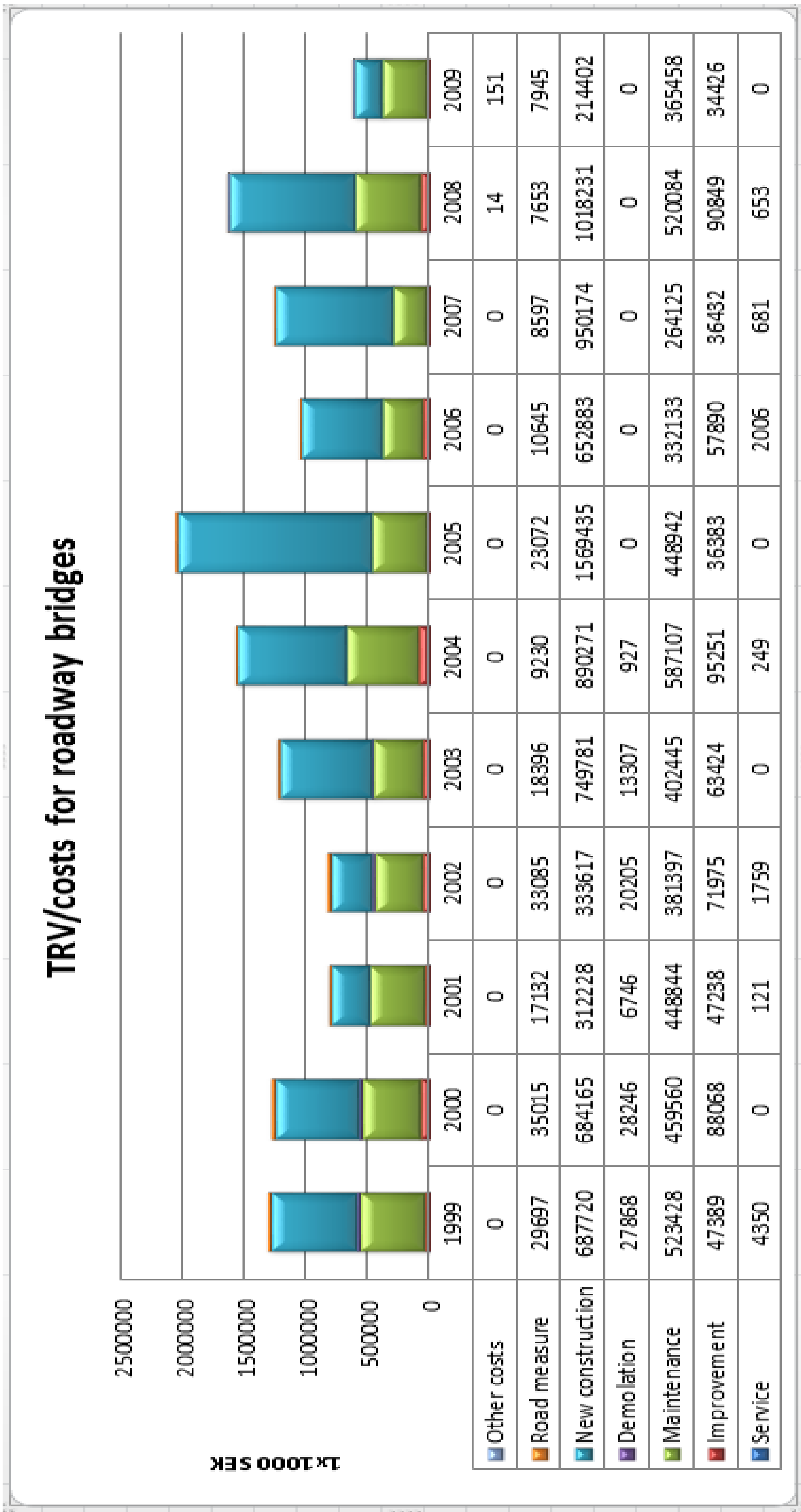
	1999	2000	2001	2002	2003	2004	2006	2008	2009
Improvement	0	10454	2928	2779	3534	1133	0	0	0
Maintenance	51431	102217	67206	87275	27165	80974	10838	943	3610

Figure A1.2 Costs made by Stockholm commune [16].



	1999	2001	2003	2004	2005	2006	2007	2008	2009
Improvement	0	0	9440	3559	88420	17970	0	4343	0
Road measure	0	0	157	743	0	0	0	0	0
New construction	0	23557	13970	14480	17548	2006	1107	14395	248375
Maintenance	661	1010	3267	1162	2474	4136	6999	5557	2307

Figure A1.3 Costs made by Trafikverket for railway bridges [16].



	1999	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009
Other costs	0	0	0	0	0	0	0	0	0	14	151
Road measure	29697	35015	17132	33085	18396	9230	23072	10645	8597	7653	7945
New construction	687720	684165	312228	333617	749781	890271	1569435	652883	950174	1018231	214402
Demolation	27868	28246	6746	20205	13307	927	0	0	0	0	0
Maintenance	523428	459560	448844	381397	402445	587107	448942	332133	264125	520084	365458
Improvement	47389	88068	47238	71975	63424	95251	36383	57890	36432	90849	34426
Service	4350	0	121	1759	0	249	0	2006	681	653	0

Figure A1.4 Costs made by Trafikverket for roadway bridges [16].

Appendix 2, Communes

Table A2.1 Communes registered in BaTMan [16].

Name list of communes in BaTMan

Alingsås kommun	Kristianstads kommun	Stockholms stad
Avesta kommun	Kristinehamns kommun	Storstockholms Lokaltrafik
Botkyrka kommun	Kungälvs kommun	Strängnäs kommun
Danderyds kommun	Köpings kommun	Sundsvalls kommun
Eskilstuna kommun	Leksands kommun	Sunne kommun
Falu kommun	Lerums kommun	Säffle kommun
Forshaga kommun	Lidingö stad	Södertälje kommun
Gävle kommun	Lindesbergs kommun	Tidaholms kommun
Göteborgs Hamn	Linköpings kommun	Torsby kommun
Göteborgs stad	Ljusnarsbergs kommun	Tranemo kommun
Hagfors kommun	Luleå kommun	Täby kommun
Halmstads kommun	Lunds kommun	Uddevalla kommun
Haninge kommun	Malmö stad	Umeå kommun
Helsingborgs stad	Mölndals stad	Upplands-Väsby kommun
Huddinge kommun	Nacka kommun	Uppsala kommun
Hällefors kommun	Nora kommun	Vallentuna kommun
Härryda kommun	Norrköpings kommun	Vänersborgs kommun
Inlandsbanan	Norrtälje kommun	Västerås stad
Jönköpings kommun	Nybro kommun	Växjö kommun
Kalmar kommun	Partille kommun	Ängelholms kommun
Karlskrona kommun	Piteå kommun	Örebro kommun
Karlstads kommun	Sigtuna kommun	Örnsköldsviks kommun
Katrineholms kommun	Skellefteå kommun	Östersunds kommun
Kils kommun	Skövde kommun	