

D 2.1

Refurbishment of existing concrete and steel-concrete bridge structures





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Colophon

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1 Executive summary

This report presents deliverable D2.1 from Work Package 2, defined in the contract documentation as ‘Construction processes for refurbishment and upgrading of existing bridge structures’.

The goal of this report is to identify common problems and technical solutions related to existing bridge structures, with main focus on road bridges. The investigation of current requirements and drawbacks of existing methods would be a foundation for developing innovative solutions in Work Package 3.

The work in this report has been carried out from November 2015 to January 2016. The responsible partner for deliverable 2.1 was CTH. The approach to achieve the goal of this report was to study the literature with focus on bridge refurbishment cases and analyze relevant processes to identify the technical difficulties experienced in the course of the studied projects.

Besides the information obtained from the literature, industrial partner AIC contributed in this report with its substantial experience in the field of diagnostics, monitoring and refurbishment of bridges by presenting some case studies.

1.1 Increasing need for rehabilitation

The need and concern for rehabilitation, upgrading and modernization of existing bridges has been increasing since 1970s. During three decades after the Second World War, construction of bridge structure boomed in Europe. A large number of bridges were built, including reconstruction of bridges that were destroyed during the war and construction of new bridges to meet the needs for increasing transportation. For instance, in Belgium with a relatively small country area, 210 bridges were built annually in the nineteen seventies [1].

After 1970s the need and concerns of bridge maintenance and rehabilitation became increasingly important. It was realized that many bridges were in need for major repair or strengthening to meet the needed capacity for passing loads and safety. However, in many cases, the real cost of bridge rehabilitations goes beyond the fund of national budgets. In the United States, the total cost needed to repair deficient bridges has been estimated to be more than 75 billion dollars, while the present budget is only about 6-7 billion dollars annually [1].

In Europe, although the situation is varying in different counties, it was found that the number of bridges that are functionally inadequate is larger than those with structural inadequacy. It means that the rehabilitation needs resulted from functional obsolescence of bridge structures, for instance, the insufficient width of roadway or demands for heavier traffic loads, are more urgent than those due to structural deficiencies [1].

Regarding general needs emphasized above, efforts are made to address the related concerns. The first version of computerized bridge management system (BMS) was developed during the nineteen eighties. In Sweden, the national BMS is named bridge and tunnel management system (BaTMan),



which helps the management of 4785 registered bridges (updated on July 9th 2015) in the nation. The BMS is also considered to be an effective tool for optimization of rehabilitation strategies and activities by allocating the maintenance fund in a more economical way [1].

An overview of bridge stocks in European countries shows that the concrete bridges account for about 60-70% of the total number of bridges in general. Thus, the need for rehabilitation of concrete bridges is more common and has higher priority for bridge owners. Due to this, the research on strengthening and repair methods of concrete structures has got more emphasis in scientific communities [1] and a wide range of structural solutions has been proposed by researchers.



2 Introduction

2.1 Aim

The aim of this report is to identify the most common problems and measures in existing bridges when it comes to maintenance and refurbishment. A critical review of the existing measures would provide a basis for development of new measures or improvement of existing ones. The information provided in this report will be utilized to avoid the problems associated with current measures when developing the concept to be developed in WP3.

2.2 Approach

The approach to obtain the goal of this deliverable was:

- (1) Identify different bridge types and their structural systems. Even though the focus of SUREBRIDGE is on concrete bridges, for the sake of completion, steel-concrete composite bridges are briefly discussed. Steel bridges, are also discussed and classified. The motivation for introduction and discussion of steel bridges is the fact that replacement of corroded steel decks in such bridges with lightweight FRP decks is a pretty straightforward rehabilitation action and has attracted attention in recent years.
- (2) Collect data from the literature and the industrial partner, AIC, regarding the most common problems in concrete bridges structures. A great deal of information was collected from the Swedish Bridge and Tunnel Management system (BaTMan [www.batman.vv.se]).
- (3) Collect data regarding the experience of industrial partner AIC in area of diagnostics and monitoring of bridges.

2.3 Limitations

Regarding the classification of bridges (Chapter 2), the analysis of common damages (Chapter 3) and the diagnostic methods (Chapter 4), the discussion is mainly focused in the scope of concrete bridges, while steel and steel-concrete composite bridges have not been discussed thoroughly. The investigation of the existing technical solutions (Chapter 5) is limited to concrete bridges and concrete components in steel-concrete composite bridges.



3 Classification of bridges regarding structural system

3.1 Concrete bridges

Concrete bridges with different types of reinforcement are discussed in this section, while the classification by construction method, span length, and static scheme can be found in Appendix 1 with detailed discussion. Due to the low tensile strength of concrete material, unreinforced concrete is rarely used for the construction of bridges and thus most of concrete structures are classified in reinforced concrete and pre-stressed concrete groups. The embedded reinforcement steel bar enhances the performance of the concrete, but it can be subjected to deterioration during the lifetime of the structure mostly due to corrosion. In order to take advantage of the concrete material, which is much stronger in compression than in tension, pre-stressing of the concrete can be utilized which can mainly be achieved by pre-tensioning and post-tensioning of the reinforcement. An advantage of pre-stressed concrete structures is that they offer smaller height compared to RC structures and thus provide maximum clearance. Further details can be found in Appendix 1.

3.2 Steel bridges

As mentioned in the introduction, the main aim of SUREBRIDGE is to develop an innovative refurbishment solution for concrete bridges using FRP decks and reinforcement of the load bearing members using FRP composite materials. However, replacement of existing corroded steel decks in steel bridges can be regarded as a solution. In this case, the new lightweight and durable FRP deck would provide a cost effective solution. For this purpose, a brief introduction and classification of steel bridges is presented in this section. Steel bridges of different types are characterized with regard to their structural system such as steel girders, truss systems, cable stayed systems. Seven types of steel bridges are introduced with details in Appendix 1.

The steel-concrete composite bridge, including a bearing system of steel girder and a concrete deck is highlighted in the scope of the report. In some cases, the steel girders can be replaced by trusses in order to realize a longer span length of the bridge, as shown in Figure 1.

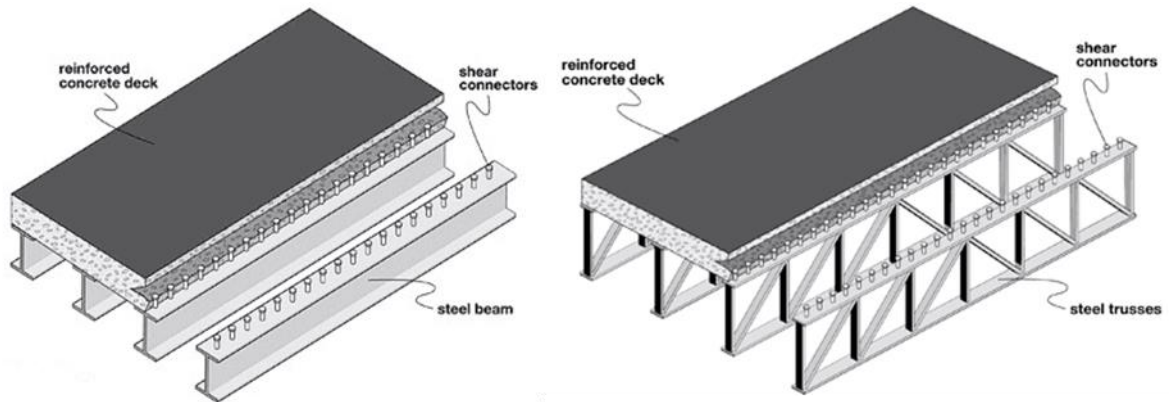


Figure 1 Illustration of a steel-concrete composite bridge. (Left: steel beam with concrete deck; Right: steel truss with concrete deck) [2]

The connection of the steel girder and the concrete deck is of great importance for the composite action between these two. Shear connectors can be used to achieve the composite action. Welded studs or bolts can be used as shear connectors, see Figure 2.

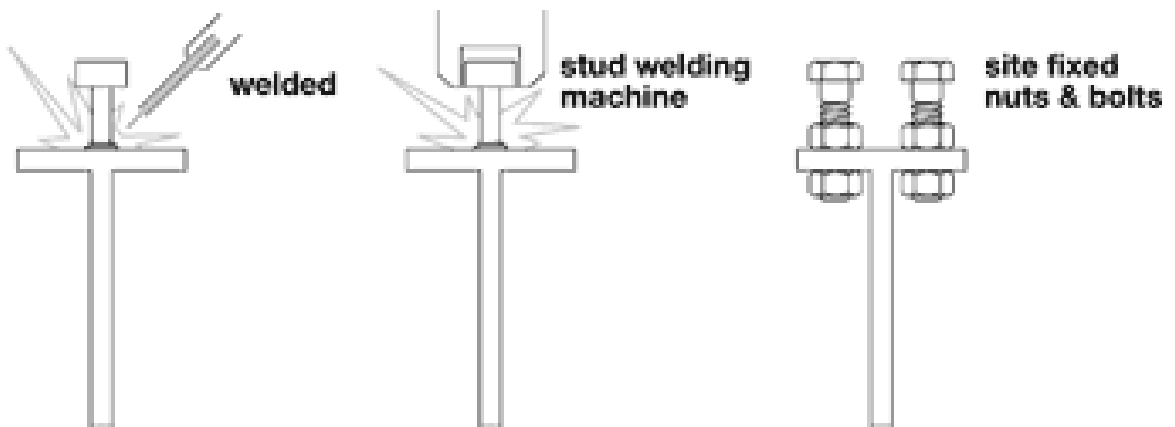


Figure 2 Methods of joining shear connector [2]



4 Material degradation and structural damage

The main causes of material degradation and structural damage are briefly summarized and discussed with reference to real case studies in this Section. In the present context, it may be useful to make a distinction between the terms “material degradation” and “structural damage”.

Material degradation refers to an alteration of the original properties of the materials used to build a construction (in particular, a bridge) over time. Material degradation is generally diffuse over an extended portion of the construction. It can be produced by aging, environmental effects (exposure to water, light, heat, ultraviolet radiation, etc.), or manmade causes (normal use, maintenance operations, etc.).

Structural damage is defined as a reduction in the original service performance or load bearing capacity of a structure. It is usually associated to localized defects (for instance, the settlement of a pier, the breakage of a bearing device, the propagation of a crack, the plasticization of a cross section, etc.). Structural damage may date back to manufacturing defects or be produced by accidental events during the structure’s service life (including loading the structure beyond its bearing capacity and errors during maintenance or refurbishment interventions).

In any case, material degradation and structural damage are closely related. Structural damage may be the ultimate consequence of material damage. In turn, structural damage may promote further material damage, and so on, until the complete failure of the structure. Ascertaining the exact causes of degradation and damage for an existing structure is the starting point for a correct design of any refurbishment or strengthening intervention. Such causes can be classified as **intrinsic**, *i.e.* due to the structure itself, and **extrinsic**, *i.e.* coming from the external environment.

4.1 Typical damage of reinforced concrete bridges

4.1.1 Intrinsic causes

Intrinsic causes of material degradation and structural damage for reinforced concrete bridges include: a) errors in design and b) errors in the construction phase.

Among errors in design, incorrect design of foundations should be cited as one of the most common causes of severe structural damage and failure of existing bridges. According to AIC’s experience in Italy, in the 1950s and 1960s, the difficulties in predicting the future settlements of a structure led many designers to use statically determined schemes (in particular, simply supported beams) to avoid structural damage resulting from differential settlements. This is the case of most road bridges on Italian highways designed up to the 1970s. Unfortunately, this design approach does not avoid damage of secondary elements, which may compromise the serviceability of the bridge.

A correct design should ensure that all primary and secondary elements have sufficient strength. Furthermore, non-structural elements should also be properly considered. For example, a popular



static scheme used for road bridges in the last century was the Gerber beam. In contrast with continuous beams, a Gerber beam has internal hinges located in cross sections between the external supports. In this case, undersizing of the internal hinges, along with inadequate design of rainwater removal systems, may produce damage such as the situation shown in Figure 3.



Figure 3 Bridge on a canal at Bientina (Pisa), Italy – Road S.R.T 439 km 51: a) Breakage of the internal hinge of the Gerber beam; b) Provisional support structure built by the Administration

Poor design of construction details is a common cause of degradation and damage. In particular, severe damage may take place by insufficient or missing rainwater removal systems. A frequent example is the insufficient length (or in some cases the complete lack) of discharge pipes from the deck, which causes corrosion of the reinforcement of edge beams, with consequent severe reduction of the load bearing capacity of the structure (Figure 4 and Figure 5).

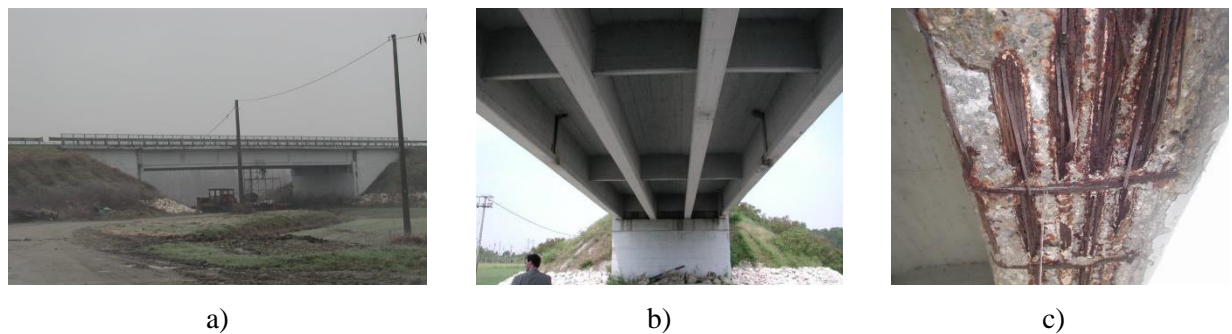


Figure 4 Bridge on the Po river at S. Nazzaro (Lodi), Italy – Road S.P. 27: a) Side view; b) Bottom view; c) Detail of corroded reinforcement bars on an edge beam. The bridge was later refurbished by adding external pre-stressing cables and a suitable rainwater removal system. The intervention was carried out in 2003



a)



b)



c)



d)

Figure 5 Bridge on the Cecina river (Pisa), Italy – Road S.P. 17 km 4+149: a) Bottom view; b) and c) Details of corroded reinforcement bars on edge beams; d) Narrowing of the roadway imposed by road Authorities

Other design errors worth mentioning, in order of importance, include the undersizing of expansion joints, the incorrect choice of bearing devices, and the insufficient concrete cover thickness. Moreover, in particular in old bridges, the lack of a suitable waterproofing may also be relevant (Figure 6). In this respect, it should also be noted that an incorrect choice of materials for the waterproofing layer may lead to debonding of the bituminous road pavement, which may slide over the deck because of high thermal excursions. Lastly, design issues include the lack (or inadequacy) of concrete mix design, which may produce degradation due to increased porosity.



a)



b)

Figure 6 Bridge on the La Lama River (Taranto), Italy – Road S.P. 13 km 9+700: a) View of the bridge; b) Severe degradation of the bottom surface of the deck due to the lack of suitable waterproofing

Moving on to **errors in the construction phase**, incorrect casting and curing methods are quite common. Defects due to improper casting are usually highly visible in the form of “honeycomb” patterns, produced by missing or insufficient vibration. Defects due to improper curing are less striking, but visible in the form of micro- and macro-cracks, which may favor penetration of aggressive agents.

4.1.2 Extrinsic causes

The interaction of concrete with the environment produces chemical reactions and physical-mechanical actions, which may be classified as follows.

Main chemical causes of degradation include:

- attack from carbon dioxide;
- attack from sulfates;
- attack from chlorides.

Main physical-mechanical causes of degradation include:

- freeze-thaw cycles;
- shrinking;
- impact, erosion, fatigue.

Physical-mechanical actions include also traffic loads higher than estimated in the project (for instance, all the bridge decks built in Italy immediately after the Second World War were designed for civilian loads only, *i.e.* a lane of trucks weighing up to 120 kN or a single roller (“Normale” No. 1 of 9 June 1945 – Italian Ministry of Public Works).

The degradation of concrete with nucleation of cracks allows external agents (water and air) to penetrate inside. This results in corrosion of the steel reinforcement bars, due to an oxidation reaction



between iron and oxygen. Since the newly formed iron oxide, or “rust”, has a specific volume higher than the specific volume of iron, as the corrosion of steel advances, portions of the concrete cover may be ejected. This produces further damage and degradation.

Typical damage phenomena involved in concrete bridges are fully introduced in Appendix 2.

4.2 Typical damage of pre-stressed concrete bridges

Pre-stressed concrete bridges suffer from the same damage and degradation phenomena described for reinforced concrete bridges.

In post-tensioned concrete beams, the inadequate design specifications of the grouting process should be mentioned among design errors as a possible cause of corrosion of the cables. Additionally, one of the main issues encountered in pre-tensioned concrete bridges is the small thickness of concrete cover that allows the corrosion of reinforcement to develop quickly (Figure 7). The reduction of wire diameter caused by rust, even of a few millimeters, can produce high additional stress in the damaged section, consequently increasing the risk of failure.



a)



b)



c)



d)

Figure 7 Bridge on the Elsa river at San Miniato (Pisa), Italy: a), b) Views of the bridge; c) and d) Details of corroded steel cables



4.3 Typical damage of steel bridges and steel-concrete composite bridges

Steel-concrete composite bridges usually show less degradation issues compared to reinforced concrete bridges. This can be ascribed to:

- 1) the special care in the design and construction that steel structures require, including special attention for construction details;
- 2) the increased attention given by road authorities in implementing a regular maintenance for this type of bridges.

Degradation phenomena observed in steel elements are generally due to:

- corrosion;
- wear in moving parts;
- fatigue;
- brittle fracture;
- element delamination.

Degradation phenomena of concrete elements are the same as described for concrete bridges.

Typical damage of steel bridges or steel-concrete composite bridges, such as corrosion, fatigue and brittle fracture, are introduced in this section. Others issues related to fasteners, joints, railings, bearings, poor detailing and external factors are included in Appendix 2.

4.3.1 Corrosion

Due to the nature of steel, typical problems that may occur in steel bridges are related to corrosion in sensitive locations. Common types of corrosion are categorized as:

- (1) Surface corrosion, leading to the uniform destruction of relatively large structural steel surface and loss of cross-sections of structural members;
- (2) Pitting corrosion occurring on small surfaces, developing deeply inside the steel and leading to local stress concentration;
- (3) Crevice corrosion, occurring in the contact layer between two elements of the same type of steel and causing destruction by tear forces resulting from swelling effects of corrosion products;
- (4) Galvanic corrosion, usually occurring in the joint location of two different types of steel or metals and leading to local material destruction;
- (5) Stress corrosion, occurring mostly in the cables in suspension bridges, usually not common in structural elements of bridges constructed of carbon steel.

The pitting corrosion, crevice corrosion and stress corrosion are also considered as fatigue corrosion. Further details of fatigue types and initiation mechanisms are introduced in Appendix 2.



4.3.2 Fatigue

Fatigue cracking is a type of damage that often cannot be avoided during the service life of steel components in bridges. In most cases fatigue cracks initiate in regions that are subjected to high local stresses, and is often associated with local defects in the material or the structural element. Fatigue is considered to be an irreversible damage mechanism. Further details of the causes for fatigue damage and examples of fatigue failure can be found in Appendix 2.

4.3.3 Brittle fracture

The term brittle fracture refers to failure of steel material at very low strains or basically failure without displaying ductility. Usually, the main reason of brittle fracture in steel is attributed to very low service temperature. However, changes in the structure of the material, by for example welding or heat treatment, might result in brittle failure mode in steel components too.

Brittle fracture results in unpredicted loss of strength and endangers the safety of the structure. This type of failure is also very expensive to repair. It is especially critical when the structure has no signs of fatigue cracking. Brittle failure usually takes place without any warning and at stress level below the ultimate strength of the material.



5 Damage diagnostics systems and methods

The diagnosis of an existing structure is the collection of experimental data and numerical analyses that allow for establishing the causes producing the observed material degradation and structural damage. The type of refurbishment intervention requires an accurate diagnosis of the structure, and evaluation of its safety. The work to be performed on an existing structure, whatever its architectural value, will be much more appropriate as the knowledge of the construction increases.

Conducting diagnostic tests is essential to define the strength and quality of the construction materials utilized. There are no international standard codes suggesting a survey protocol, but in Italy there are several guidelines addressing the survey procedures. To obtain a complete picture of data, tests should be performed on all structural elements. However, due to economic limitations, this is often not feasible. Therefore, development of a survey program for the execution of the tests is necessary, according to the analysis performed in a pre-diagnostic phase. The survey program should preferably be prepared by the same designer, who will assume the responsibility of making the diagnosis and, consequently, the proposals for the action. The tests should also be aimed at ensuring complete exploitation of the survey results. More generally, the survey must first of all be able to: (i) match the hypotheses formulated by the designer, (ii) provide physical and mechanical parameters to be used in structural analysis and, if necessary, (iii) provide control data for the structure at given intervals of time (monitoring). The tests to be carried out on site should be mainly non-destructive. To this aim, sophisticated techniques, such as those based on the transmission of elastic or electromagnetic waves, can be used. Unfortunately, non-destructive techniques often require high costs and a calibration for each specific application.

5.1 Diagnostic process

The diagnostic process is accomplished through the following steps:

- 1) Pre-diagnosis
 - a) Visual survey, conducted with naked eye or with only aid of low magnification factor lenses or endoscopes; it is used to determine the main characteristics of materials, structures, construction details, and a preliminary identification of the states of degradation and damage;
 - b) Geometric survey aimed at determining the shape and size of each relevant element of the construction;
 - c) Structural survey, with particular attention on construction techniques, construction details, and connections between structural elements;
 - d) Survey of damage, understood as a full description of any degradation, cracking, and deformation phenomena;
 - e) Investigation of the subsoil and foundation structures, with particular reference to the changes that have occurred over time and the related damage;



- f) Retrieval of the original design documentation (technical reports, drawings, specifications, test certificates, certificates of materials, etc.).
- 2) On site investigations
 - a) Destructive testing: mainly, mechanical tests (extraction of samples, surface tension test, compression of cores and micro-cores);
 - b) Non-destructive testing: mechanical tests (sclerometric, penetration, and static load tests), sonic tests (ultrasonic, micro-seismic tests, acoustic tomography, acoustic emission), endoscopy (photo- and video-endoscopy), electromagnetic tests (X-ray, X-ray tomography, thermography, radar surveys, electromagnetic surveys, corrosion potential), chemical tests (depth of carbonation, chloride content), combined tests (two- and three-parameters SonReb tests, SonReb + Windsor), other tests (laser scanning, etc.);
 - c) Dynamic identification tests: acquisition through accelerometers of the natural frequencies and modal shapes of the structure undergoing free vibrations triggered by natural (environmental) or artificial excitations; acquisition of the response of the structure subjected to dynamic loads, e.g. produced by instrumented hammers or vibrodynes. The results of dynamics tests can be used to calibrate models used for structural analysis and safety checking.
- 3) Laboratory tests on materials and samples extracted from the structure.
- 4) Simulated design: following the rules in effect at the time of construction with reference to loads, calculation models, requirements on materials, executive details, etc., a simulated design can be conducted to deduce some construction details that have not been (or cannot be) determined by on-site investigations and are presumably present in the analyzed structure.
- 5) Structural analysis
 - a) Schematization of the structural problem;
 - b) Definition of calculation models using the data derived from the surveys along with the on-site and laboratory tests;
 - c) Analysis of the models with the finite element method; possibly, calibration with the above described dynamic acquisitions;
 - d) Control of results and safety checking;
 - e) Results of safety checking;
 - f) Summary of the results to identify the relevant structural issues and the intervention criteria.
- 6) Design of strengthening intervention.



5.2 Visual inspection

The visual inspection is the first step necessary for the condition assessment of structures. By means of visual inspection, an overall impression should be obtained for all symptoms of deterioration including the identification of actual and potential sources of trouble. All the activities leading to the final choice of a rehabilitation strategy for a damaged bridge are initiated at this stage [3].

In order to ensure an optimum collection of information, an inspection concept is needed, including [3]:

- Description of the structure
- Historical information (including previous inspection reports)
- Access equipment needed (tower wagon or scaffold) – lane closure, necessary downtimes
- Possible removal of everything that prevents good visual access
- Inspection equipment
- Competences and responsibilities

Inspections which may significantly interfere with normal traffic movement and affect the safety of the inspectors must be coordinated with district personnel in order that appropriate traffic control measures may be undertaken. Inspections of the underside of bridges that cannot be reached by conventional ladders may be performed by the use of vehicles with under-bridge platforms [3].

During the visual inspection special attention needs to be paid to various factors, including:

- Verification of information gathered during the planning of the assessment
- Old coatings, impregnations or protections
- The appearance of the original concrete surface
- Differences of the color of the concrete surface
- The presence of cracks, their appearance and pattern
- Superficial deterioration of the concrete skin
- Deterioration of the concrete itself
- Exposed reinforcement bars
- Deformations of the structure
- Presence of humidity or water, leakages, etc.
- Fouling (algae, moss, tresses)

The findings must be described in detail as they form the basis for any consequent measures. In addition, it may be useful to state the name of the inspector, as well as the names of all that may have been attendance [3].

5.2.1 Routine inspection

The routine inspection is a visual inspection of all visible parts of the structure. The inspection should be carried out by a highly experienced bridge engineer. The purpose is to maintain an overview of



the general condition of the whole infrastructure stock, and to reveal significant damage in due time, taking safety and economic aspects into consideration.

Common concrete member defects include cracking, scaling, delamination, spalling, efflorescence, wear or abrasion, collision damage, scour, and overload. The inspection of concrete should include both visual and physical examination.

Common steel and iron member defects include corrosion, crack, collision damage, and overstress. Cracks usually initiate at the connection detail, at the termination end of a weld, or at corroded location of a member and then propagate across the section until the member fractures. Since all the cracks may lead to failure, bridge inspectors need to look at each and every one of these potential crack locations carefully [3].

Further introduction of routine inspection regarding the result of condition ratings and equipment needed is included in Appendix 3.

5.2.2 In-depth inspection

In depth inspections are usually performed as a follow-up inspection to a Routine Inspection to better identify any deficiencies found. A testing program for reinforced concrete structures that supplements visual observations may include obtaining and testing cores and samples for compressive strength, chloride ion content, carbonation depth, pH value, and petrographic examinations. Load testing may also sometimes be performed as part of an In-Depth Inspection. [3] Further details about testing of concrete and steel strength, carbonation depth, and chloride content can be found in Appendix 3.

5.3 Non-destructive evaluation (NDE)

Assessing the structural condition without removing the individual structural components is known as non-destructive evaluation (NDE) or non-destructive inspection. NDE techniques include those involving acoustics, dye penetrating, eddy current, emission spectroscopy, fiber-optic sensors, fiber-scope, hardness testing, isotope, leak testing, optics, magnetic particles, magnetic inspection, etc. Most of these techniques have been used successfully to detect location of certain elements, cracks or weld defects, corrosion/erosion, and so on [4].

The most common NDE techniques used for concrete evaluation, reinforcement detection and steel evaluation are emphasized in this section. Further details are included in the Appendix 3.

5.3.1 NDE of concrete

Concrete structures could deteriorate due to heavy traffic loads, fatigue, chemical reactions, unpredictable disasters, and poor workmanship [6]. In this regard, effective inspection techniques become very important prior to repair works. The principal NDE techniques for concrete evaluation discussed in Appendix 3 include:



- (1) Radiographic testing for the detection of broken wires in cable-stayed bridges, imaging of post-tensioning strands in concrete beams, and the detection of voids in the grouted post-tensioning ducts.
- (2) Ultrasonic pulse echo test, ultrasonic pulse velocity tests and impact echo testing for identifying the location of cracks and other internal defects in the concrete.
- (3) Infrared thermography test, a non-contact optical method to detect a) voids in grouted masonry, b) delaminations in concrete bridge decks and slabs, and c) moisture variations in roof and wall systems.
- (4) Ground-penetrating radar (GPR) systems, a pulse-echo method for measuring layer thickness and other properties.
- (5) Acoustic Emission monitoring to detect cracking, delamination, cleavage, and fretting in a material [5].

5.3.2 Reinforcement detection

As a result of the rapid development in non-destructive testing during the last decade, several reliable methods have been proposed to determine the location and diameter of steel reinforcement. Three detection methods – electro-magnetic method, radiography method, and radar method – and their benefits and limitations are described in Appendix 3.

5.3.3 NDE of steel

The principal NDE techniques for steel evaluation discussed in Appendix 3 include:

- (1) Penetrant testing used to locate the surface defects in non-porous material, such as cracking and porosity in welded joints, surface defects in castings, and fatigue cracking in stressed materials.
- (2) Magnetic particle testing used for crack identification.
- (3) The Eddy-Current Method for testing the quality of welds and detect residual stresses in objects of any shape.
- (4) The radiographic method used to inspect the quality of butt welds in the fabrication of steel plates for bridge girders.
- (5) The ultrasonic method for locating the discontinuities in the evaluated components.



6 Existing technical solutions for repair and strengthening of bridges

The rehabilitation method generally consists of selection of proper material and suitable technique for execution. Generally, existing rehabilitation methods can be categorized into measures taken for structural repair and structural strengthening. In this section, a general overview of repair and strengthening methods of concrete structures is presented. Due to the advantages offered by fiber reinforced polymer (FRP) composites in strengthening of existing structures, this technique is more elaborated in this section.

6.1 Existing methods for repair

6.1.1 Measures and structural solutions for repair of bridge structures

To select proper repair techniques and material solutions, it is necessary to consider the criteria including the location of the damaged area and accessibility. The common types of repair work in the concrete and steel-concrete composite structures are listed in the Table 1.

Table 1 General classification of standard repair techniques and materials applied to concrete bridge superstructures [1]

Type of work	To be repaired	Techniques	Repairing material
Removal of deteriorated concrete	Concrete; All structural members	Hand chipping, Pneumatic hammer, Saw cutting, Water jet	--
Corrosion removal	Reinforcement steel; Anchorage; Steel bearings or joints;	Hand removal, Sandblasting	--
Surface cleaning	Concrete; Steel	Hand washing, Jet blast of air or water	--
Crack repair	Concrete	Surface coating, Injection	Cement or epoxy
Bonding the repair material	Concrete	Hand techniques	Cement mortar or adhesive
Patching	Concrete	Hand techniques	Cement mortar or concrete



Replacement or addition of the reinforcement	Reinforcing steel	Hand techniques, Welding	Steel reinforcing bars
Reinforcement protection	Reinforcing steel	By hand, Spraying	Epoxy coating
Applying the repair material	Concrete	Cast by gravity, Shotcrete,	Cement mortar, concrete
Surface coating and sealing	Concrete	Hand techniques	Sealants for surface sealing; polymer or cement for surface coating
Repair of collision damage in the structural members	Mostly reinforced or pre-stressed concrete beams or box girders	Post-tensioning, Internal splicing, External splicing	Pre-stressing tendons

6.1.2 Crack repair

Concrete is known as a material having very good compressive strength but weak strength characteristics when it comes to tension. Therefore, cracking is very common in concrete structures due to a variety of reasons. The critical factors of the structural performance, however, is not the existence of cracks, but their width, depth and locations. In terms of the consequence, the cracks are divided into two groups: 1) cracks resulting in a negative influence on the durability, for instance, causing a leakage of the structural element without reducing the load bearing capacity, and 2) cracks causing weakening of structural element [1].

In group 1, the cracks can be repaired by surface coating (crack widths less than 0.2 mm) or by crack sealing with a proper filling material (crack widths more than 0.2 mm). Resin or cement-based material injection can be applied by gravitational or pressure injection processes to repair cracking in the concrete. In group 2, the structural parts separated by the cracks are repaired by an adhesive or a bonding material with higher strength than the base material [1]. Further details about injection methods and crack filling materials are introduced in Appendix 4.

6.1.3 Surface repair

In general surface repair includes filling up of concrete loss in the structure or replacement of the deteriorated concrete by various repair materials. Depending on the deterioration depth of concrete, the methods can be divided into shallow repairs and deep repairs. If the depth of concrete deterioration is less than the concrete cover, then shallow repair is applied. The application of deep repair happens when the deterioration depth is greater than the concrete cover and the steel reinforcement is exposed. Especially in the bridge deck, a total-depth repair is applied when the deteriorated concrete depth exceeds half of the total deck thickness [1].



The general procedure for conventional surface repair in concrete bridges usually includes:

- The removal of deteriorated concrete
- Cleaning of the exposed concrete and removing the corrosion on steel reinforcement
- Anticorrosion protection of steel reinforcement
- Application of bonding coat and placing the repair material
- Finishing and surface protection

Cement based mortar and concrete are the most widely used repair materials. With the development of repair materials, the fiber reinforce concrete and polymer cement concrete are getting more attention for this purpose and are increasingly used in contemporary repair projects. The selection of techniques to apply the repair materials depends on the scale of the repair activity, location of the structural surface to be repaired, and the type of repair material. Typical techniques include [1]:

- Placing the concrete by hand technique
- Casting by gravity
- Shotcrete
- Pumping

Further information about the surface repair procedure and the techniques used for applying repair material is presented in Appendix 4.

6.2 Existing methods for strengthening

Table 2 lists the strengthening solutions and divides them into passive and active methods. In the category of passive methods, the application of strengthening elements can lead to certain redistribution of the internal forces, but the redistribution itself is not the purpose of these methods. Instead, active strengthening methods aim to introduce a redistribution of internal forces in the structure. In this section, conventional solutions by using steel plates or reinforcement and the bonded FRP solutions are underlined. The potential to develop a lighter superstructure members with FRP composite materials is also discussed.

Table 2 Methods for bridge strengthening [1]

Passive methods

Enlargement of cross-sections of the structural members by additional reinforced concrete layers

Steel plate or steel flat bonding and/or bolting (external plating)

External bonding of fiber reinforced polymer (FRP) strips

External bonding of wrap composite fabrics

Replacement of some structural members by new ones (especially precast concrete elements)

Active methods



Redistribution of the internal forces in transversal direction by additional loading of internal beams or girders and unloading of the external ones

Installation of the additional strengthening members, e.g. steel truss girders

Lightening of the superstructures by replacement of some concrete structural members, e.g. by steel or FRP members

Pre-stressing by additional tendons, e.g. strengthening by external post-tensioning

Strengthening by pre-tensioned FRP strips

Installation/Substitution of bearing devices to modify the distribution of support reactions

Installation/Substitution of seismic isolators, dampers, etc. to modify the dynamic response

6.2.1 Reinforcement of concrete decks

A common technique used to increase the structural performance of damaged concrete decks is to cast a new additional concrete layer. This usually involves the following sequence of operations (Figure 8):

- 1) milling the top layer of the existing slab (this operation should be preferably conducted by high-pressure water-jetting, as using a motor wrecker could damage the existing steel reinforcement);
- 2) installation of connectors;
- 3) placement of new steel reinforcement bars;
- 4) casting a new (light-weight) concrete layer.

A drawback to this technique is that the addition of a new layer to the deck, despite using lightweight concrete, leads to an increase of structural weight and mass. This unavoidably alters the bridge's static and dynamic behavior. The consequent increase in stress and deformation might not be compatible with the strength of other structural elements of the bridge (including its foundations), which may require further reinforcement.



a)



b)



c)



d)

Figure 8 Bridge “Attilio Vergai” at Villa Collemantina (Lucca), Italy: a) View of the bridge; b), c) and d) Placing of connectors and reinforcement bars for the new light-weight concrete layer. The intervention was conducted in a 3 month timeframe in 2009, with the bridge closed to traffic.

6.2.2 Strengthening by lightening the superstructure

Replacement of the original concrete superstructure with lighter structural members is an active strengthening method. The principle is based on the fact that the total internal forces are resulted from an algebraic sum of the dead loads and live loads. Thus, reducing the dead load on the bridge by using lighter superstructure can lead to an increased capacity regarding the live loads [1].

In general, this method is appropriated for the strengthening of continuous beam systems with intermediate hinges. Steel girders or decks with smaller cross-section and less weight are normally used to replace the concrete elements. However, the construction time needed on site is rather long. Considering the development of FRP materials, a competitive idea is motivated to replace the old concrete deck with an FRP one, which has excellent strength to weight ratio. Due to its light weight, it is also possible to reduce the installation time required in the strengthening operation, resulting in less disruption of traffic and lower user cost.



6.2.3 Strengthening by using externally bonded steel plates or FRP strips

A common method for strengthening of a bridge span is bonding of external steel plates, see Figure 9. Steel plates are applied in areas where the tensile strength is insufficient because of a deficiency of the reinforcement necessary to carry the traffic load. Application of bonded steel plates can increase the flexural strength and load carrying capacity of bending elements, such as beams and slabs, and also the compressive strength of columns. More details about the conventional steel plates bonding are included in Appendix 4.

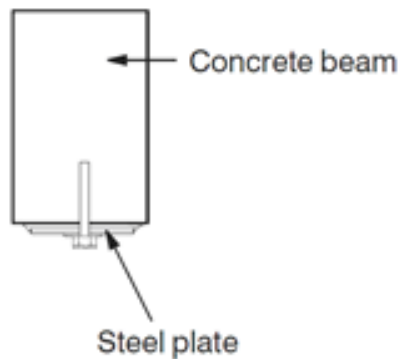


Figure 9 Scheme of an externally bonded steel plate to a concrete beam

With development of FRP materials, the application of FRP strips instead of steel plates has become a common practice. It is possible to use fiber-reinforced polymers (FRP) in the shape of laminates. Compared with the conventional steel plates, the FRP composite has good fatigue properties, excellent corrosion resistance and high strength-to-weight ratio in terms of material properties.

Several types of fiberfiber are available on the market to this aim: glass, aramid, and carbon fiberfibers, but also vegetal fiberfibers, which have gained interest in recent years due to environmental concerns. For bridges, carbon fiberfiber-reinforced polymers (CFRP) are mostly used because of their superior mechanical performance. In some cases, laminates are also applied on the bottom surface of the deck.

The stress transfer between the FRP laminates and concrete elements enables structural collaboration between the two materials, giving rise to a new composite structure. The quality and strength of the interface bond are essential requirements to increase the bearing capacity of the structure. Actually, failure of these reinforcement interventions is typically due to delamination, i.e. de-bonding, of the FRP laminate from the concrete support. In particular, fracture may involve one of three components: concrete, adhesive, FRP (Figure 10). In general, the limited shear strength of concrete (in particular, if degraded) constitutes a real “drawback” for this type of intervention.

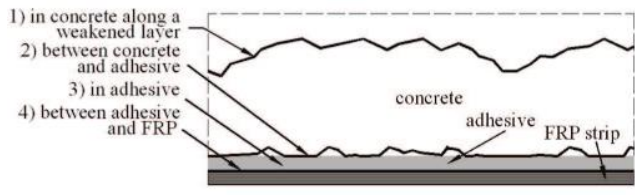


Figure 10 Interface fracture of FRP laminates bonded to a concrete substrate

Besides, Figure 11 shows the difference of these two solutions regarding installation process indicating the ease of application in case of FRP strengthening. The comparison and analysis of the two methods is presented in Table 3.

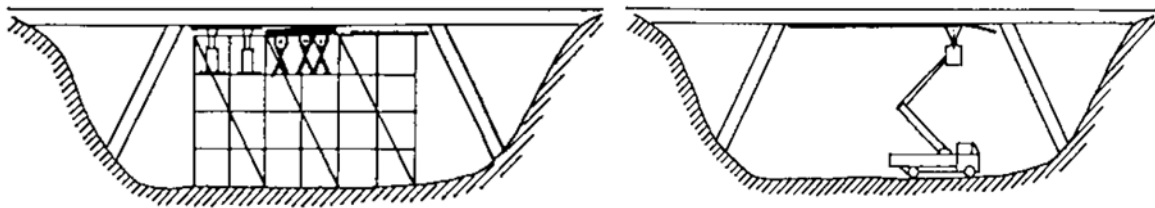


Figure 11 General comparison of the use of two analogous bridge strengthening methods [1] (Left: Strengthening by means of steel plate bonding; Right: strengthening by means of CFRP strip bonding)

Table 3 Advantages and disadvantages of steel plate bonding and FRP strips bonding [1]

Steel plate bonding	FRP strips bonding
Advantages	
<ul style="list-style-type: none"> • Relatively low material cost • Commonly used • Sufficiently high strength, also fatigue strength • Load bearing in any direction • Possibility of the use of bolt or screw anchorages if necessary 	<ul style="list-style-type: none"> • No corrosion problems • Very light weight • Very high strength and fatigue resistance • Easy handling on the construction site • No length limitation • Low labor cost • Possibility to avoid scaffolding cost • Application with no special tools
Disadvantages	
<ul style="list-style-type: none"> • Corrosion problems • Relatively high weight • Some handling difficulties on construction site • Limited length (joint problems) • High scaffolding cost • High labor cost • Application with lifting gear and clamping device 	<ul style="list-style-type: none"> • Relatively high material cost • Only bearing load in longitudinal direction



6.2.4 Strengthening with pre-stressed FRP composites

Different from the passive FRP strengthening method introduced above, the concept of using pre-stressed FRP is developed in order to utilize the composite material more efficiently. The pre-stressing can be accomplished by pre-tensioning or post-tensioning methods. The popular types of FRP materials identified by researchers include aramid fiber reinforced polymers (AFRP), carbon fiber reinforced polymers (CFRP), and glass fiber reinforced polymers (GFRP) in the form of rods, strips, plates and laminates [6]. CFRP is the most popular one for the strengthening of concrete structures. Depending on different application techniques, three strengthening methods using FRP materials are introduced in this report. Further details of externally bonded FRP system and examples of FRP application process can be found in Appendix 4.

6.2.4.1 Pre-stressed FRP in externally bonded technique

In the externally bonded reinforcement (EBR) technique, FRP plates or laminates are pre-stressed and bonded to the external surface of the strengthened beam using epoxy adhesives. Pre-stressing the FRP plates or laminates prior to bonding allows the high tensile strength of the material to be exploited, which results in improvement in the service load range of the structure.

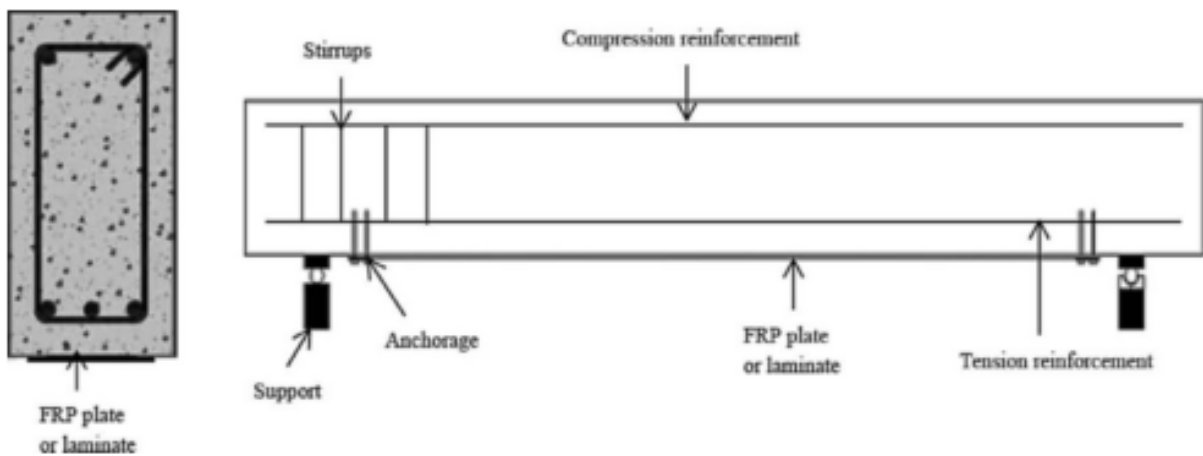


Figure 12 Externally bonded pre-stressed FRP reinforcement [6]

The pre-stressing force induces compressive strain in the bottom side of the section (i.e. tension zone), resulting in an upward camber that reduces the deflection of the member throughout the loading as shown in Figure 12.

6.2.4.2 Pre-stressed FRP in near surface mounted technique

The near surface mounted (NSM) method was initially presented in 1940 [7]. Steel cables were applied but the problem with high corrosion led to replacement of steel cables with FRPs. In the NSM technique, pre-stressed FRP rods or strips are inserted into grooves on the concrete surface and bonded to the concrete using epoxy adhesive, see Figure 13. Flexural strengthening using pre-stressed NSM FRP reinforcement can increase the ultimate strength of an RC member quite



remarkably. It also significantly changes the behavior of the member under service loads and substantially increases the stiffness of the member.

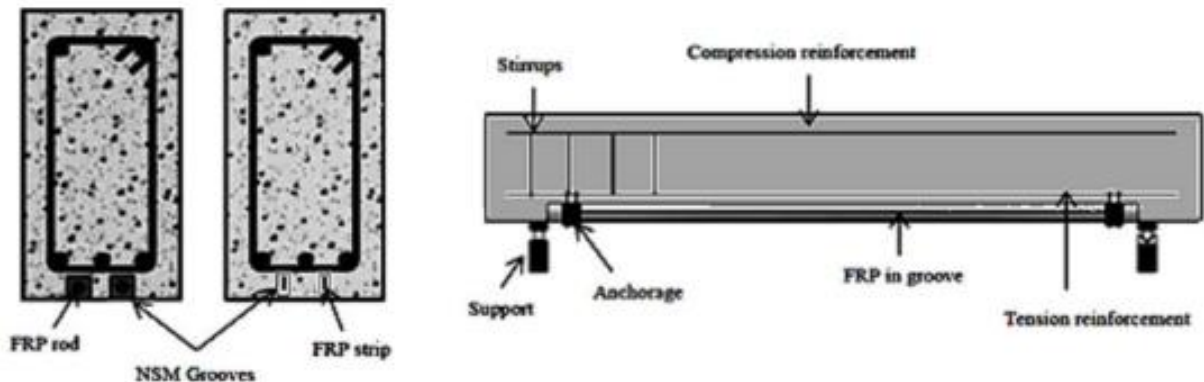


Figure 13 Schematic diagram of near surface mounted pre-stressed FRP reinforcement [6]

Pre-stressed FRPs using the NSM technique can reduce the ductility of the strengthened beam [8].

Benefits of using pre-stressed composites compared to passive applications [8]

- Improved serviceability of the beam.
- Reduced dead load deflections.
- Reduced crack widths and delayed start of cracking.
- Relieved strains in the internal steel reinforcement.
- Increased yielding of internal steel reinforcement at a higher proportion of the ultimate load.
- More efficient use of the concrete and the FRP materials.

Comparison of NSM and EBR

The performance of strengthened RC beams by NSM CFRP strips was compared to the CFRP strips externally applied on the tension face of concrete beams at different pre-stressing levels by Aslam et al. (2015) [6]. Generally, the NSM technique is more effective and gives relatively better results in terms of ultimate load capacity, due to higher bond strength at the concrete-FRP interface [9]. The FRP composites in NSM technique are also safer regarding the external mechanical damage and fire, since these are placed in the grooves on concrete surface and are completely covered by the epoxy adhesive. However, the downside with this technique is the need for modification of the concrete surface in terms of cutting grooves which is time consuming and costly.

6.2.4.3 FRP in externally post-tensioned technique

The externally post-tensioned technique was initially introduced for bridge strengthening in 1950s, see Figure 14. Nowadays, this technique is applied for both deficient and newly built structures.

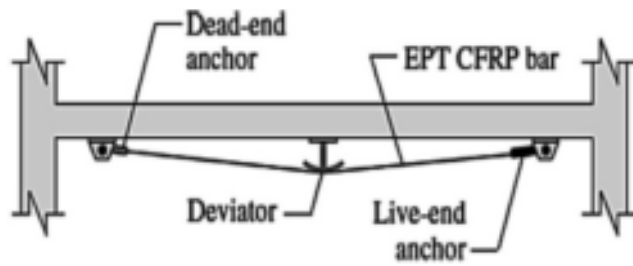


Figure 14 Pre-stressing system for externally post-tensioning CFRP & Steel (Rods) [10]

In the case of strengthening using this technique, an increase in structural performance can be achieved by introducing an external pre-stressing (Figure 15). Post-tensioning is an economic, adaptable, and effective system. Also, the external bars or cables may be easily inspected and, if necessary, substituted. Conversely, this technique has some disadvantages related to durability (the external cables are exposed to atmospheric agents) and realisation (anchoring devices made of steel may be quite heavy and require special cranes, etc.).



Figure 15 Bridge on the Taro River at Solignano (Parma), Italy – Road S.S. 308 km 11+850: a) View of the bridge; b) Detail of an external cable anchoring device made of steel. The project included reinforcement of the bridge's piers, deck, and longitudinal beams. The intervention was performed with one-way traffic open during 2003 - 2004).



Advantages and disadvantages

Some advantages for using external pre-stressing for both new and existing structures [11], [12]:

- The dimensions of the structural section can be reduced due to less space required for the internal steel reinforcement.
- The assembly of the external steel tendons are easier and simpler to check during and after installation.
- The external steel tendons can be replaced and removed if the corrosion protection of the tendons leads to a release of the pre-stressing force.
- The frictional losses are considerably reduced because the external steel tendons are only connected to the structural member at the deviation and anchorage zones.
- The main structural operations, pre-stressing and concreting, are more independent of one another; therefore, the effect of workmanship on the overall quality of the structure can be reduced.

The following are some of the disadvantages that should be kept in mind [11], [12]:

- The external post-tensioned tendons are more easily accessible than internal ones and, subsequently, are more exposed to damage and fire.
- The external post-tensioned tendons are exposed to vibrations and, consequently, their free length should be adequate.
- In the deviation zones, high transverse pressure acts on the post-tensioned steel tendon. The deviation zones should be properly mounted to decrease the friction as much as possible and to avoid damage to the pre-stressed steel tendon.
- At the ultimate limit states, the flexural strength is reduced due to the external tendons as compared to the internal tendons.
- The stress variations between the cracking and ultimate load may not be assessed at the critical section only, as is done for internal tendons.
- At ultimate limit states, the main concern for the externally pre-stressed structures is the failure with little warning due to inadequate ductility.

The following are some of the advantages of NSM over EBR and EPT [6], [8], [13]:

- Excellent for strengthening in the negative moment regions, where EBR would be subjected to mechanical and environmental damage.
- Feasibility of anchoring into members adjacent to the one to be strengthened.
- Less likely to debond near ultimate capacity.
- Protection of the embedded FRP in the grooves from external damage, such as vehicle impact, better fire performance, resistance to moisture, and avoiding of freeze–thaw problems.



- The choice of FRP material with higher strength and modulus of elasticity, such as CFRP instead of GFRP and AFRP, would allow the use of smaller FRP and groove cross sectional areas; hence, there is less risk of interfering with the internal reinforcement.

In terms of structural behavior, the most relevant mechanical properties are the tensile and shear strengths; therefore, the grooves can be properly filled in with epoxy adhesive or cement mortar.

6.3 Demolition and reconstruction of structures

In some cases, when degradation and damage severely compromise the structural integrity, or when the Client's demand for an increased bearing capacity are very high, demolition and reconstruction of the concrete deck can be considered as a solution. As a last resort – also considering financial aspects and provided that the existing bridge has not a historical or monumental value – demolishing and rebuilding of the whole bridge may also be considered.

As an example, this solution was chosen for a road bridge over the Arno river in the town of Empoli (Figure 16). The Client's request was to double the traffic lanes of the existing bridge, whose structural performance was severely compromised. A new bridge was therefore constructed next to the existing one with new piers and their foundations (the central span was about 50 m). Then, a first deck having a carriageway with two vehicular lanes was constructed. Afterwards, the existing bridge was demolished, and finally the second deck in the exact location of the existing one was realised. The proposed solution had no disadvantage to the current flow capacity of the bridge: two lanes remained open during the construction phases, moving from the current bridge to the newly realised deck. The most critical phase for vehicular traffic was related to the demolition of the existing bridge. In order to minimize disturbance as much as possible, the demolition was performed during the night hours.

The construction phases are:

- realisation of piers and foundations of the new bridge next to the existing ones;
- construction, close to the existing bridge, of a first deck having a carriageway equal to half of the final bridge (in other words, two vehicular lanes, one bike path and walkway);
- commissioning of the new temporary bridge deck, with one lane in each direction;
- demolition of the existing bridge;
- realisation of the second deck, symmetric to the first one, in place of the existing one;
- final realisation of the four lanes carriageway as foreseen in the project;
- construction of access viability and finishing work items.



a)



b)



c)



d)



e)

Figure 16 Bridge at Empoli (Florence), Italy: a) and b) Views of the existing bridge; c) and d) Construction of first deck of new bridge near existing; e) Completed bridge open to traffic



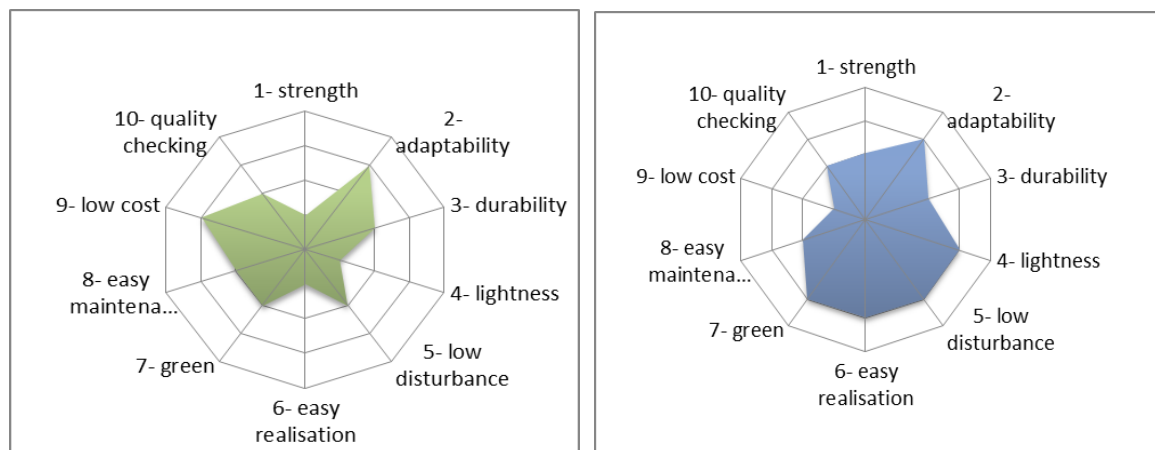
7 Conclusions

One of the most commonly adopted techniques for the refurbishment of reinforced concrete decks, in case of excessive degradation and damage to the deck, is the (partial) demolition and reconstruction (with optional additional layers) of the reinforced concrete slab. This solution, has obvious limitations related to the high level of invasiveness on the surrounding environment, long realization time with high disturbance to traffic and production of waste, dust, and noise during demolition.

The choice of the most suitable technique for the refurbishment of an existing bridge should be driven not only by structural issues, but also by an assessment of cost effectiveness over the expected lifetime of the structure. This can be accomplished by carrying out a life cycle cost analysis (LCCA). An ideal refurbishment technique should satisfy the following requirements:

- high mechanical performances (in particular, strength);
- adaptability to complex, existing geometries;
- high durability;
- light weight of the reinforcement system;
- low disturbance;
- easy and quick realization at construction site;
- high environmental sustainability;
- easy maintenance;
- low cost;
- possibility to check the effectiveness of the reinforcement system over time.

Figure 17 summarizes an evaluation graphic for the most common refurbishment techniques with scores from 1 to 3. The larger size of the areas shows the better performance. Accordingly, FRP reinforcement ranks first, followed by external pre-stressing and realization of additional concrete layer; the worse option turns out to be the demolition and reconstruction of deck.



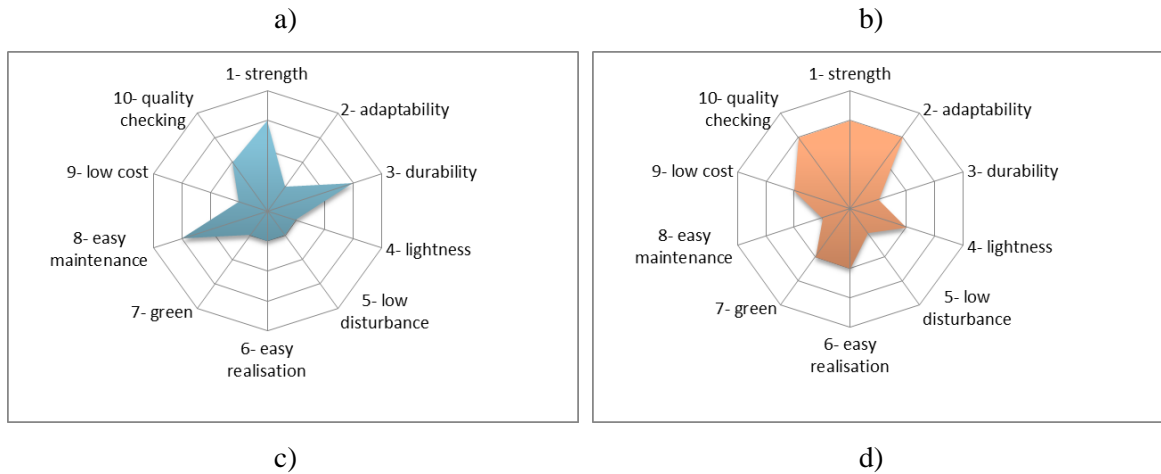


Figure 17 Evaluation of refurbishment techniques: a) Additional concrete layer b) FRP reinforcement; c) Demolition and reconstruction of deck; d) External pre-stressing.



8 References

- [1] Radomski, W., 2002, Bridge Rehabilitation. Imperial College Press, ISBN 1860941222.
- [2] REIDsteel, “Steel Bridges Design | Bridge Construction Consultants.” [Online]. Available: <http://www.steel-bridges.com/advice.html>. [Accessed: 15-Jan-2016].
- [3] Fib, Monitoring and Safety Evaluation of Existing Concrete Structures: State-of-art Report. International Federation for Structural Concrete (fib), 2003.
- [4] Wang, T. L. , Zong, Z., 2002, Improvement of Evaluation Method for Existing Highway Bridges. Miami: Florida Department of Transportation.
- [5] RILEM Technical Committee (Masayasu Ohtsu), “Recommendation of RILEM TC 212-ACD: acoustic emission and related NDE techniques for crack detection and damage evaluation in concrete*,” Mater. Struct., vol. 43, no. 9, pp. 1183–1186, Jul. 2010.
- [6] Aslam, M. , Shafigh, P., Jumaat, M. Z., Shah, S. N. R., 2015, Strengthening of RC beams using prestressed fiber reinforced polymers – A review, Constr. Build. Mater., vol. 82: 235–256.
- [7] Badawi, M., Soudki, K., 2009, Flexural strengthening of RC beams with prestressed NSM CFRP rods – Experimental and analytical investigation, Constr. Build. Mater., vol. 23, no. 10: 3292–3300.
- [8] El-Hacha. R., Soudki, K., 2013, Prestressed near-surface mounted fiber reinforced polymer reinforcement for concrete structures — a review¹, Can. J. Civ. Eng.
- [9] El-Hacha, R., Rizkalla, S. H., 2004, Near-surface-mounted fiber-reinforced polymer reinforcements for flexural strengthening of concrete structures, ACI Struct. J., vol. 101, no. 5: 717–726.
- [10] Matta, F., Nanni, A., Abdelrazaq, A., Gremel, D., Koch, R., 2009, Externally post-tensioned carbon FRP bar system for deflection control, Constr. Build. Mater., vol. 23, no. 4: 1628–1639.
- [11] Picard, A., Massicotte, B., Bastien, J., 1995, Relative Efficiency of External Prestressing, J. Struct. Eng., vol. 121, no. 12: 1832–1841.
- [12] Nordin, H., Täljsten, B. , 2006, Concrete Beams Strengthened with Prestressed Near Surface Mounted CFRP, J. Compos. Constr., vol. 10, no. 1: 60–68.
- [13] De Lorenzis, L., Nanni, A., 2002, Bond between near-surface mounted fiber-reinforced polymer rods and concrete in structural strengthening, ACI Struct. J., vol. 99, no. 2: 123–132.

D 2.1 – Appendix 1

Classification of concrete and steel bridges





D 2.1 – Appendix 1

Classification of concrete and steel bridges

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Colophon

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1 Introduction

In order to identify suitable structural solutions for refurbishment purposes, it is necessary for the designer to have a clear understanding about the structural system in the structure of interest. This appendix intends to give a brief introduction to different types of concrete and steel bridges with focus on their structural and load bearing systems. Brief information regarding construction methods is presented as well.

2 Steel bridge types

In this section different types of steel bridges and their structural systems are briefly introduced. In reality, each category is selected based on the length of the span, traffic loads, width of the bridge and architectural considerations. Choice of bridge types is limited to the studied literature.

2.1 Steel girder bridges

This type is the most common type of steel bridges. Attractiveness of this type is mainly due to its simplicity in manufacturing and construction. In normal practice the steel girders are often used together with a concrete slab to provide the roadway. In many cases, the concrete slab has composite action with steel girders resulting in an increase in the stiffness of the system. In such cases, the concrete slab help this system to cover spans up to 30 m as shown in Figure 1. It has been mentioned by some references that spans longer than 15 m are usually not economical [1]. Steel girders are also used together with steel decking systems such as orthotropic steel decks.

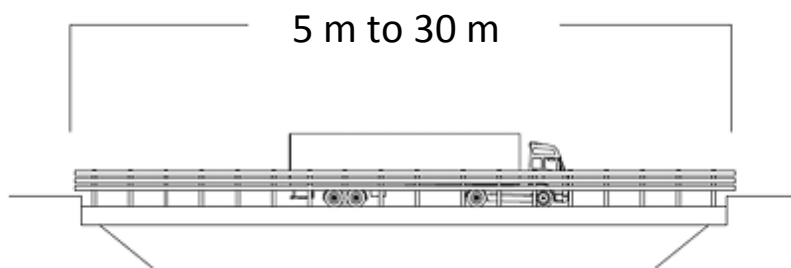


Figure 1 Range of span lengths in composite steel beam bridges [1]

An 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 2 [1].

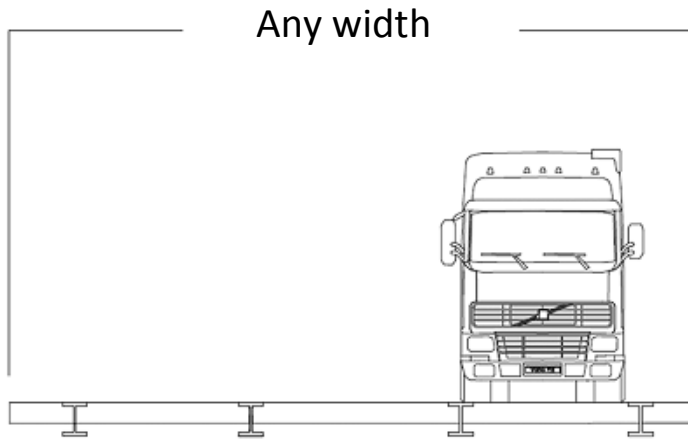


Figure 2 Flexibility in width in steel girder bridges [1]

The most common applications of steel girder bridges are in pedestrian and highway bridges. This type of bridge can be constructed in multi spans. In this case, the girders can be simply supported in each span or be continuous as shown in Figure 3.



Figure 3 Erection of continuous steel beam bridge [1]

2.2 Truss bridges

Truss systems are known to be one of the most efficient structural configurations which can offer very high stiffness to weight ratio and are thus suitable where larger spans are to be covered or the light weight of the structure can improve the logistics of the project. Truss bridges are divided into two main groups of over truss and through truss systems.

2.3 Over truss bridges

The structural system of this kind of bridges is very similar to that of steel girder bridges, but in this case trusses are used instead of steel beams. Examples can be seen in Figure 4 and Figure 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 [1].

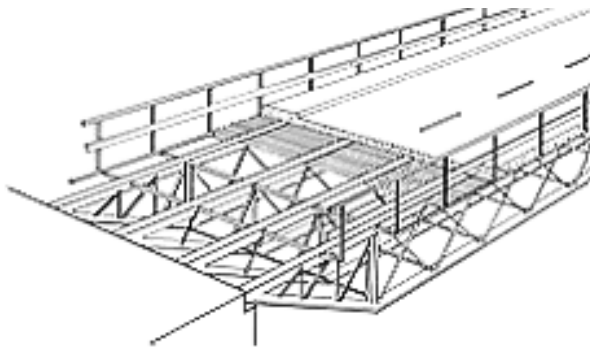


Figure 4 View of an over truss bridge [1]



Figure 5 An example of steel over truss bridge [1]

The span in this type of bridge varies between 20 m and 100 m, see Figure 6 and the width of the bridge can vary depending on the need. The construction of over truss bridges is more complex and requires more work compared to steel beam bridges [1].

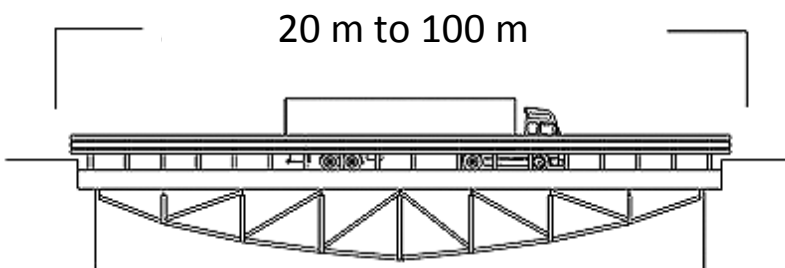


Figure 6 View of an over truss bridge with typical span lengths [1].

Construction of this bridge type can be done in-situ using three different methods as illustrated in Figure 7 [1]. This method is applicable when the bridge is to be built over a river or waterway.

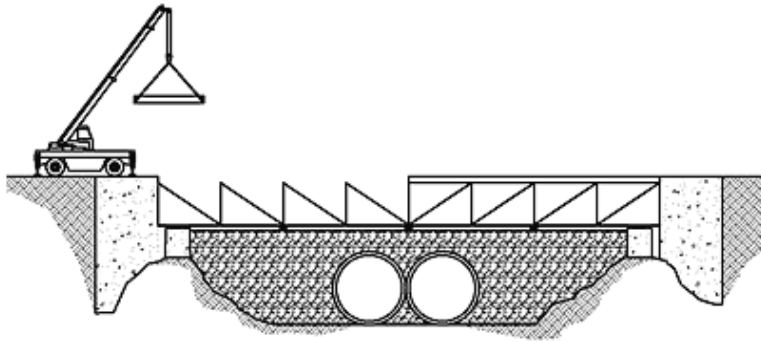


Figure 7 Construction of a trough truss bridge [1]

Another method is lifting the pre-constructed truss system, see Figure 8 [1].

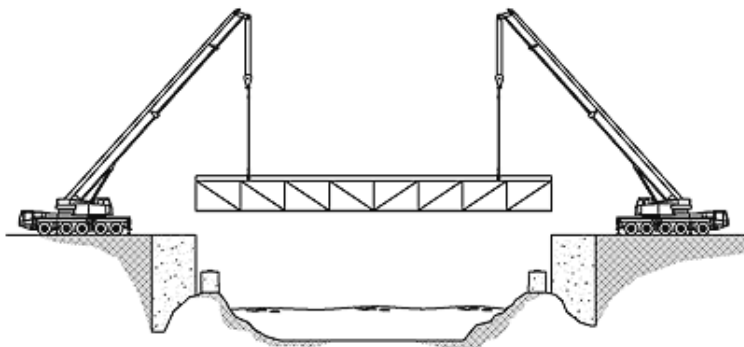


Figure 8 Lifting of constructed beams and joining at the place [1].

Roller launch method is also applicable in situations illustrated in Figure 9 [1].



Figure 9 Roller launch method [1].

2.4 Through truss bridges

Trough truss bridges are simple to design and have a modern appearance, see Figure 10.



Figure 10 Through truss bridge (name and place of the bridge are unknown) [1].

The span of the through truss bridges can be up to 120 m and the headroom does not require special level, see Figure 11. The width of the bridge varies depending on the need [1].

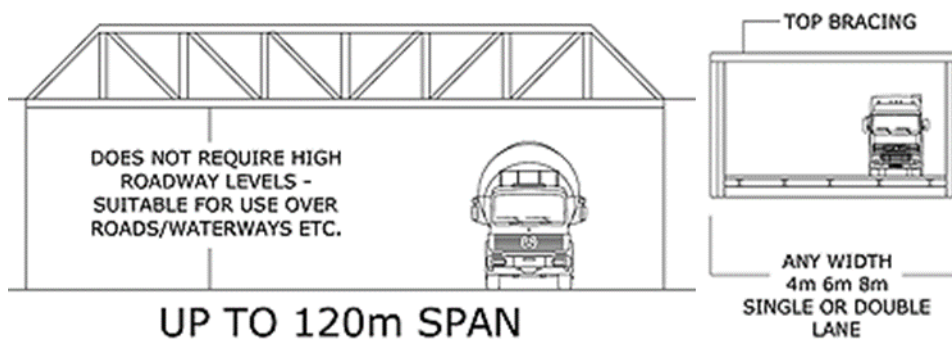


Figure 11 A typical trough truss bridge with long span [1].

The light weight and slender structure make this kind of bridges economical. The use of through truss bridges is beneficial in areas where earthquake is an issue but not recommended in windy areas [1]. Through truss bridge can be constructed in-situ assembly as illustrated in Figure 12.

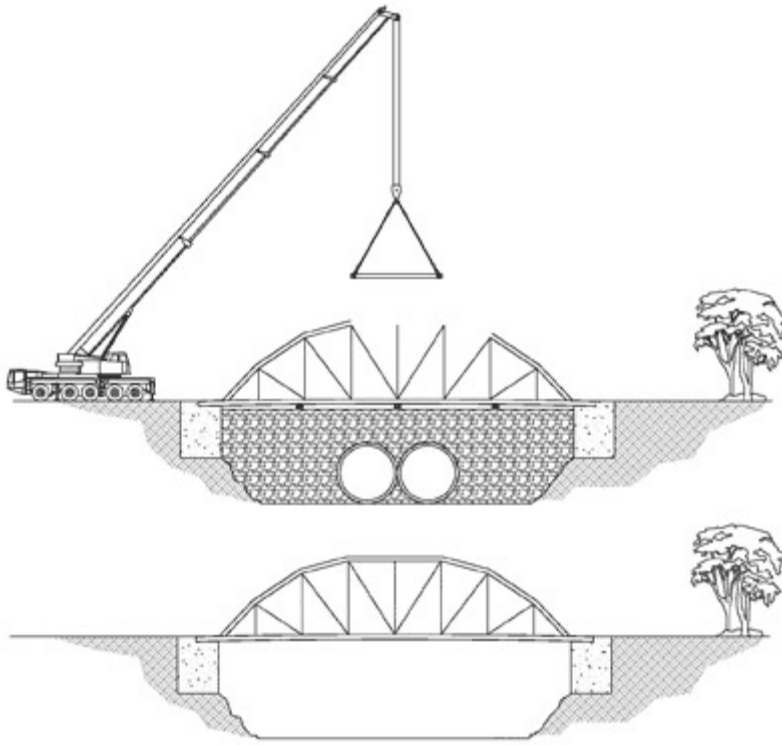


Figure 12 Erection of a trough truss bridge [1].

Another method is assembly on-site and then lifting the bridge on place, see Figure 13 [1].

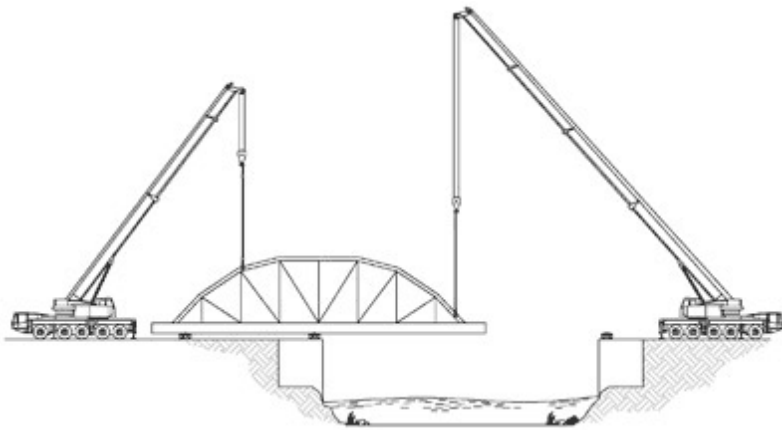


Figure 13 Lifting of the trough truss bridge to place [1].

Figure 14 shows roller launch method. When the bridge is assembled on-site and is moved by jacks with help of rollers and cantilever technique [1].

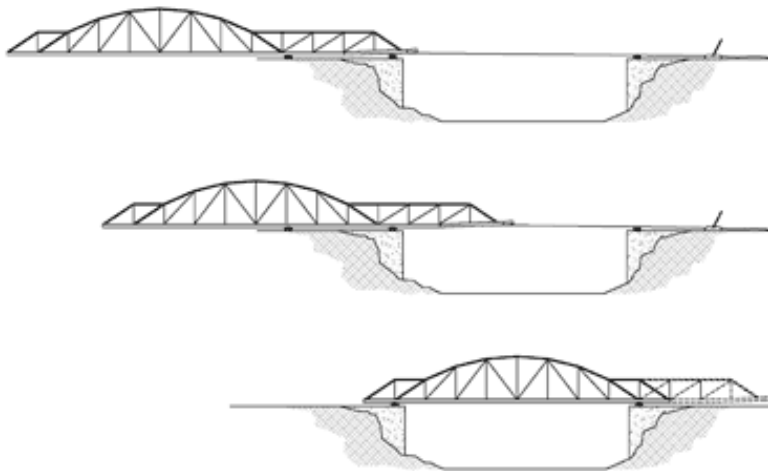


Figure 14 Roller launch method [1].

2.5 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 15 [1].

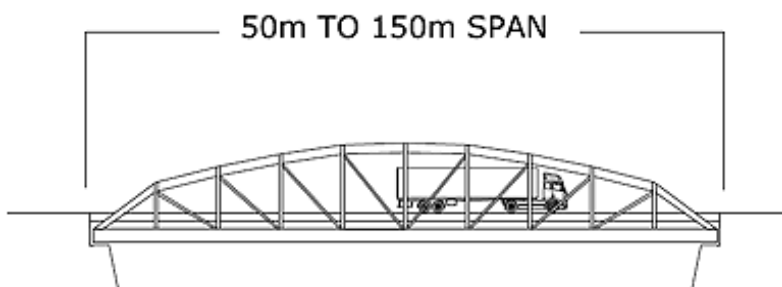


Figure 15 A view of steel bowstring girder bridge [1]

This kind of bridges take advantage of arc and tensile tie action. They are visually attractive and can be constructed on uneven soils or less stable foundations. They can be assembled on-site or alternatively be pre-fabricated and transported to the site [1]. Figure 16 shows a steel bowstring girder bridge [1].



Figure 16 Steel bowstring girder bridge in Sudan [1]



2.6 Cable-stayed bridge

Cable stayed bridges use the same principle as suspension bridges. In this type of bridge, the deck is supported by tension cables which are supported on one or more towers. The towers, act in compression and the deck girders undergo compression and bending actions. The economic span for cable stayed bridges is from 200 to 850 meters.

Cable-stayed bridges are expensive to construct. This kind of bridges is preferable when there is no space for piers and no opportunity for cantilever launching. Figure 17 shows two different models of cable-stayed bridges. The first one is a single tower cable stayed bridge in which the main span can be from 50 m to 100 m. The second one is a double (multiple) tower cable stayed bridge. The main span in this configuration can be from 100 m to 200 m, see Figure 17. Double (multiple) tower cable stayed bridges require a space of one fifth of main span as outer span [20].

Cable-stayed bridges are light weight and comprise slender structural members. These properties are advantageous in areas with risk of earthquake and disadvantageous in windy areas [1].

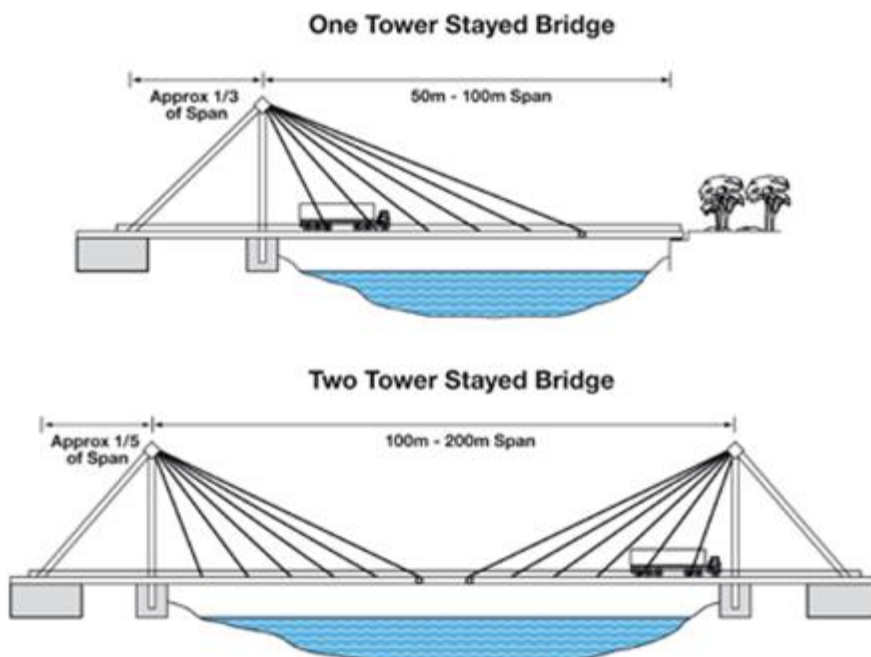


Figure 17 Two examples of cable-stayed bridge [1].

Erection of the cable-stayed bridges is illustrated in Figure 18, Figure 19, Figure 20 and Figure 21 [1].

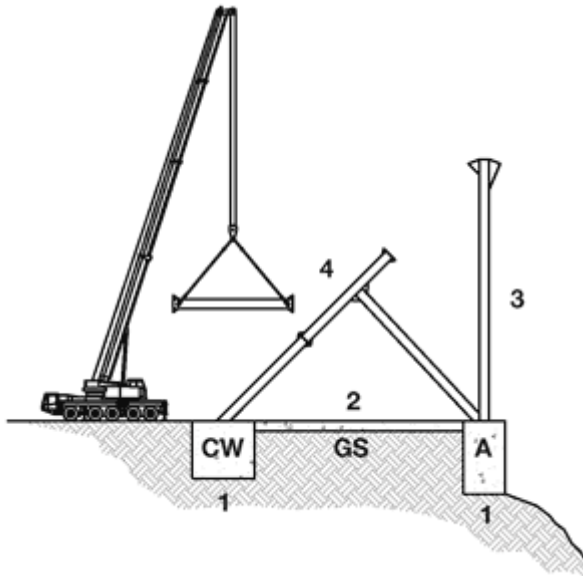


Figure 18 Erection of the abutment and the tower.

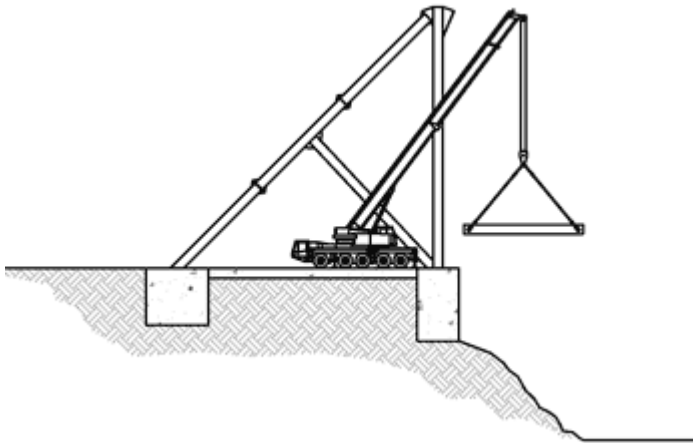


Figure 19 Continued erection by deck

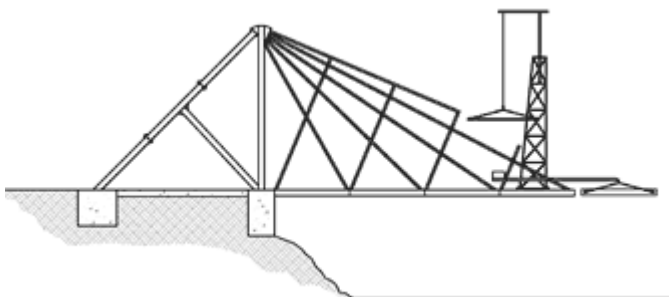


Figure 20 Erection cables

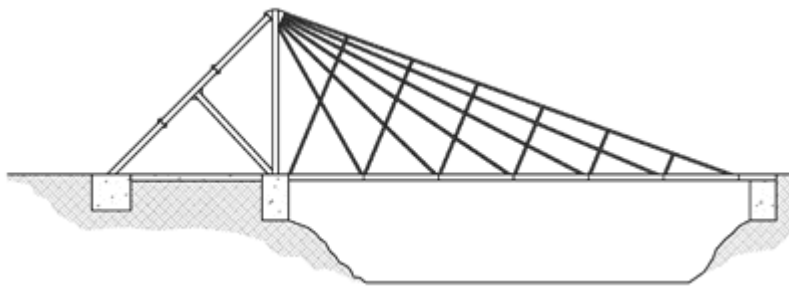


Figure 21 A view of completed bridge

2.7 Suspension bridges

In suspension bridges, the roadways suspends from main cables, which extend from one end of the bridge to the other. The main cables rest on towers and are anchored to foundations on either end of the bridge. High towers enable the main cables to be draped over long distances. Most of the weight of the bridge is transferred to the anchorage system by the cables. The anchors are embedded in solid rock or concrete blocks.

The deck of a suspension bridge is supported by vertical tension hangers, which are supported in turn by large tension cables extending over two towers from anchor to anchor, see Figure 22.

A stiffening girder running over full length of each span is an essential part of a suspension bridge. It distributes the concentrated traffic loads and provides stiffness against bending, twisting and oscillation. For single decks, the trend is to use box girders to minimize weight and give maximum torsional stiffness. However, where double decks are required (for example, to carry road traffic and railway traffic) then the general trend to use truss girders.

Suspension bridges tend to be most expensive option among other types.

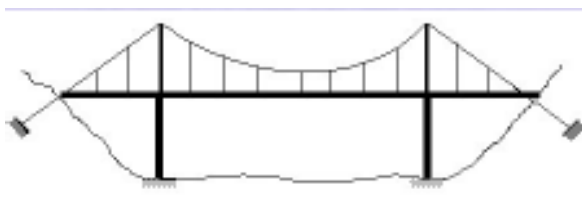


Figure 22 Schematic of a suspension bridge

2.8 Steel-concrete composite bridges

Steel-concrete composite bridges include a bearing system of steel girders and a concrete, see Figure 23. The steel structure and the concrete deck are connected to each other and in a composite manner which results in reduction of deflection and increase of strength [1].

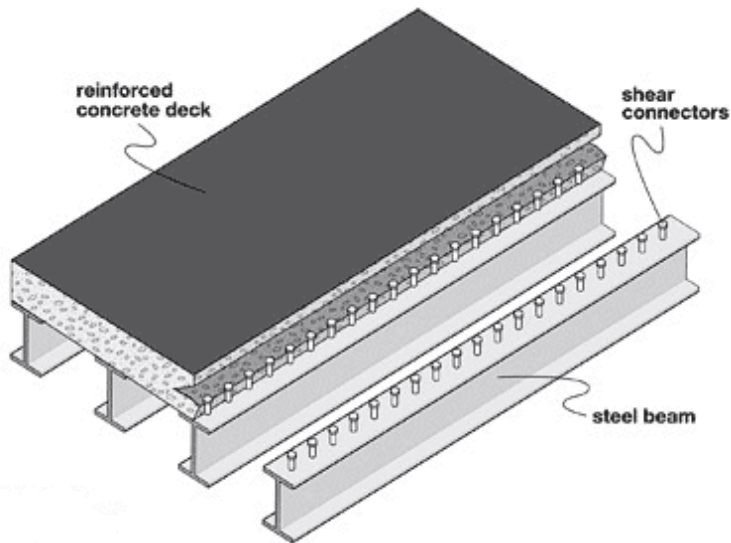


Figure 23 Illustration of composite bridge [1]

The connection between the steel structure and the deck can be achieved by shear connectors. Attachment of shear connectors may be done by welding or nuts and bolts fixed on site, see Figure 24 [1].

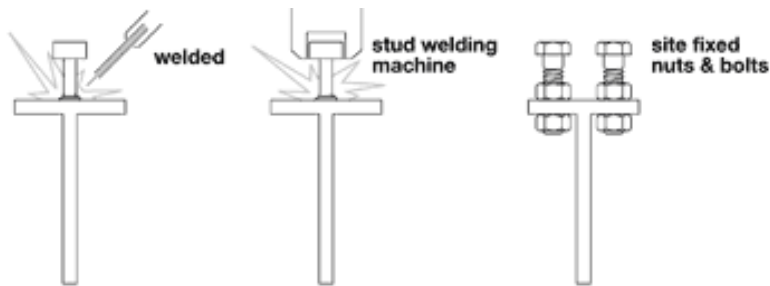


Figure 24 Methods of joining shear connectors [1]

Composite bridges can be built using steel beams or trusses. Figure 23 illustrates the case with steel beam. In this case, usually spans of 8 m to 15 m are covered. A simply supported beam of this kind can be built up to 24 m but often span lengths more than 15 m are not economical [1]. Figure 25 illustrates an over truss bridge with concrete deck. Span can vary approximately from 18 m to 100 m [1].

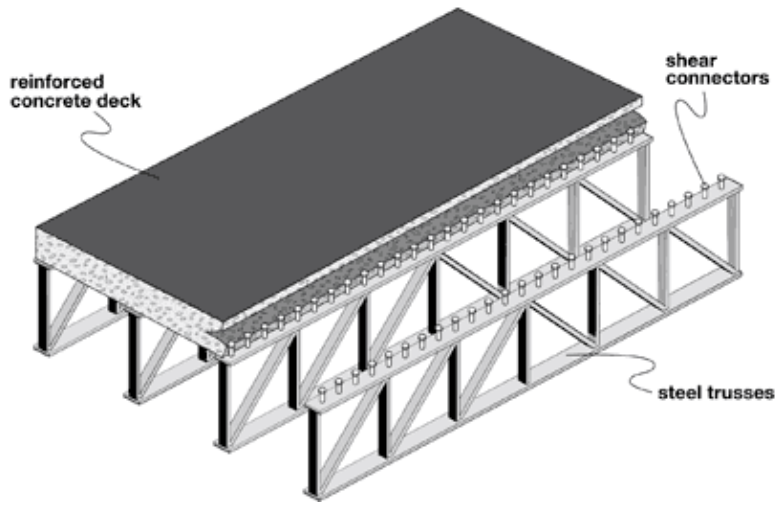


Figure 25 A view of over truss bridge with concrete deck [1].



3 Concrete bridge types

3.1 Introduction

This section introduces different types of concrete bridges and their structural systems. Concrete is the most widely used construction material in bridges in the world. Combination of high compressive strength of concrete with high tensile strength of reinforcement (steel or FRP material) enables possibility of designing and building great superstructures with different cross sections.

3.2 Classification by type of reinforcement

3.2.1 Unreinforced concrete

The unreinforced concrete is characterized by high compressive and low tensile strength. Its tensile strength varies from 8 to 14% of its compressive strength [7]. For this reason, it is rarely used for construction of bridges. Nevertheless, arch bridges made of this material can be found among historical bridges. An example, Borrodale Viaduct (Highland, Scotland, United Kingdom) built in 1901 is shown in Figure 26.



Figure 26 Borrodale Viaduct, Scotland, UK, built in unreinforced concrete (photo by Jim Bain)

3.2.2 Reinforced concrete

In order to enable the concrete to carry tensile loads, it should be reinforced with a material which has good characteristics in tension. Typically, steel bars are used to reinforce concrete. Unfortunately, steel is susceptible to corrosion causing degradation and consequent loss in load



bearing capacity of the structure. In recent years, FRP reinforcements are becoming increasingly popular. FRP rebars are usually manufactured using a two-stage process. At the first stage, the bars are pultruded to a partially cured state, to be afterwards compressed with an additional layer to provide surface lugs similar to steel bars in the second step of the process [8].

3.2.3 Pre-stressed concrete

To make even better use of the material capacity, concrete can be pre-stressed. Pre-stressed concrete is very common in bridge construction. Pre-stressing techniques allow achieving very slender bridges. The most common types of pre-stressed concrete bridges include: girder bridges, arch bridges, frame bridges, slab bridges, and cable-stayed bridges. An advantage of pre-stressed concrete bridges is that they can have smaller height in order to provide maximum clearance. Through pre-stressing it is possible to utilize the maximum span-to-depth ratio. Span-to-depth ratio as high as 35:1, or even more, can be achieved with solid slabs, voided slabs, I-beams or box beam cross sections. Deeper sections will require less pre-stressed steel. Pre-stressed concrete has much better freeze-thaw and chemical resistance compared to traditional reinforced concrete bridges. The pre-stress limit depends on the type of the cross-section and static scheme.

There are two main types of pre-stressed concrete; pre-tensioned and post-tensioned. In the first one, concrete is cast around already tensioned tendons (usually wire, strand, a bar or bundle of strands [9]). When the concrete is sufficiently bond, anchors of tendons are released and the element becomes pre-stressed (Figure 27). In the second method the concrete is cast in a mold around a curved duct (plastic, aluminum or steel – Figure 28) including the cables. As soon as it obtains appropriate strength, the tendons (in the ducts) can be tensioned by hydraulic jacks and anchored. The duct is then grouted to protect the tendons from corrosion (Figure 28). Post-tensioned concrete bridges can be realized by incremental launching or balanced cantilever methods (Figure 28). The latter method is often used to connect precast elements. It should be noticed that the elements may be pre-stressed by both methods.



Figure 27 Pre-tensioned concrete beams and their installation on the building site



Figure 28 Top left: post-tensioned concrete bridge constructed with incremental launching [6]; Bottom left: post-tensioned concrete bridge erected with cantilever method [6]; Right: cable in aluminum ducts in the formwork (photo by Störfix)

3.3 Classification by type of construction

3.3.1 Monolithic (cast in place)

Traditional reinforced concrete bridges are constructed on site (Figure 29). This kind of structure requires space for formwork or false-work (Figure 29 and Figure 30) which is a labor intensive process and contributes to a longer construction time. The main difference between scaffolding and false-work is that false-work requires larger and heavier temporary support for the superstructure. Instead of supporting the load via a large number of cross-braced struts, steel beams between temporarily established supports are used. Usually spans have a range from 10 to 20 m. False-work can therefore be used when obstacles like small rivers or roads need to be reached over. Both methods, mentioned above, are suitable to use if the free height of the bridge is not greater than 6 m.

An advantage of cast in-situ bridges is the monolithic structure and its greater durability in comparison with prefabricated concrete components. The downside, however, is the lengthy process and dependency on the weather condition.



Figure 29 On-site fabrication of reinforced concrete bridges

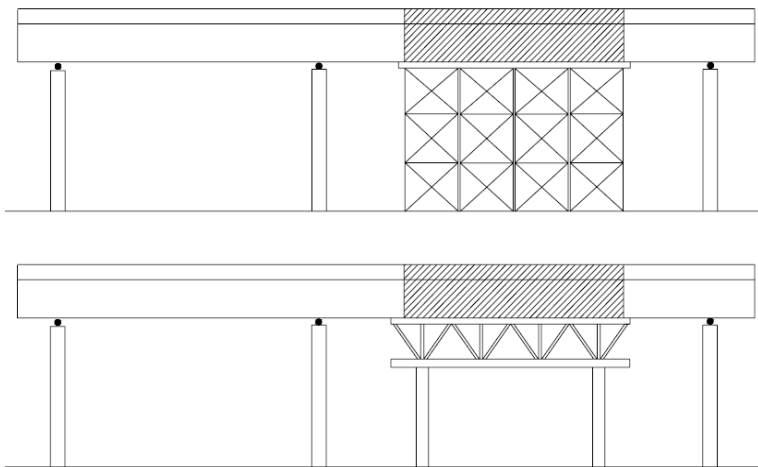


Figure 30 Top: Bridge constructed with scaffolding; Bottom: bridge constructed with false-work [4]

3.3.2 Precast (prefabricated)

Precast elements are found to be very cost-effective due to the savings associated with maximizing repetition in production in factory conditions and shortening the construction time (Figure 27). Concrete is a brittle material, so the transportation can be dangerous for prefabricated elements. To reduce this risk, precast elements are usually pre-stressed.

Typically, the weakest points in prefabricated bridges are the connections between the precast elements (due to discontinuities of material). Beams as well as plates and box sections can be produced as prefabricated elements. Figure 31 illustrates different connection configurations at the location of piers.

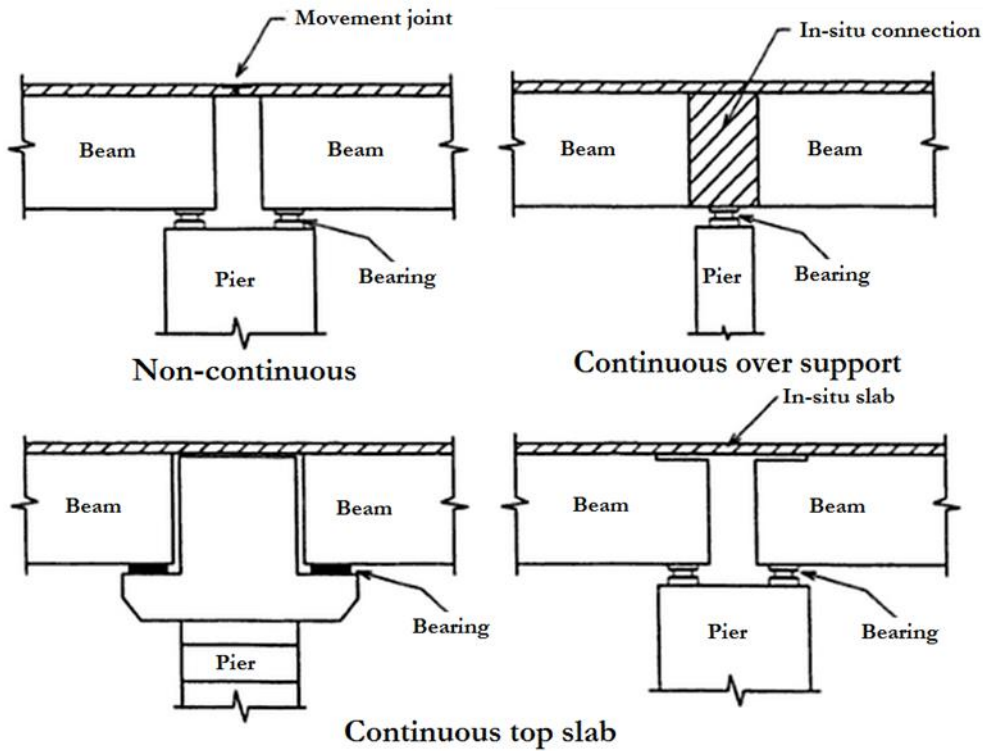


Figure 31 Different connection configurations at location of piers [9]

3.4 Classification by type of span

3.4.1 Slab bridges

The basic cross-section in concrete bridges is solid concrete slab (Figure 32). A reinforced slab bridge is the simplest type and the most common for short spans. The main advantages are: small height, freedom in placement of supports, easy forms of the scaffolding, comparatively easy placing of the reinforcement or tendons and pouring of concrete, elimination of classic bridge elements like transverse beams, bridge deck, bracings, good rigidity, and favorable transfer of unsymmetrical loads. Disadvantage is the high bridge dead-weight that increases the loads that need to be transferred to foundations. In order to minimize the dead-weight of the slab, it can have voids inside, i.e. hollow slabs.

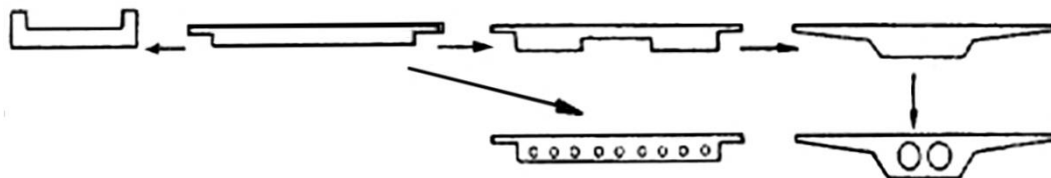


Figure 32 Examples of cross-sections in concrete slab bridges [2]



The span lengths that can be achieved with a slab bridge are fairly short, maximum span length is around 15 m, therefore, they tend to be used on small stream crossings. The superstructure depth for slab bridges generally ranges between 30 cm and 50 cm. Where site conditions require a shallow superstructure, the slab bridge is quite often the best choice. A typical reinforced concrete slab bridge is shown in Figure 33.

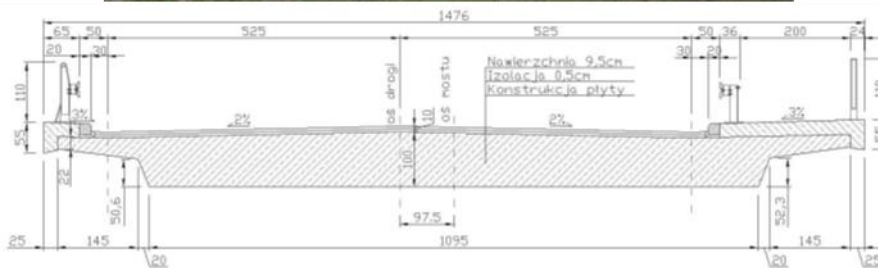


Figure 33 Concrete slab bridge in Poland [3]

3.4.2 Girder bridges

Concrete girder bridge is a common design choice for small and medium spans, in general up to 40m. The number of beams depends on the bridge width and the load type (Figure 34).

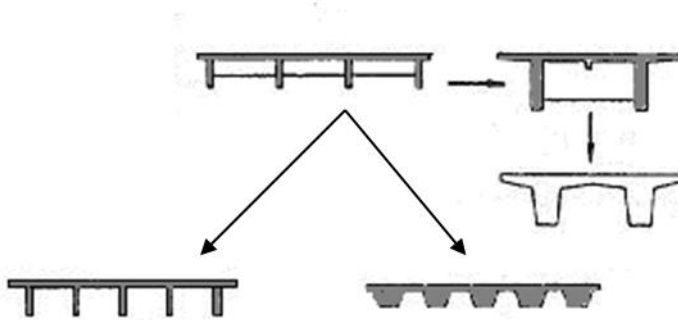


Figure 34 Cross-section of concrete girder bridge [2]



Drawback of concrete girder bridge is the difficult on-site fabrication, caused by complicated scaffolding needed for concrete casting. Therefore, this type of cross-section often utilize prefabricated components that are casted in the fabrication plant and transported to the building site.

3.4.3 Box bridges

Box bridges allow longer spans and in general are applied for a span range between 40m-250m. The cross-section can be built up from one or more boxes (Figure 35). The concrete is often pre-stressed. Advantage of such configuration is the high torsional resistance.

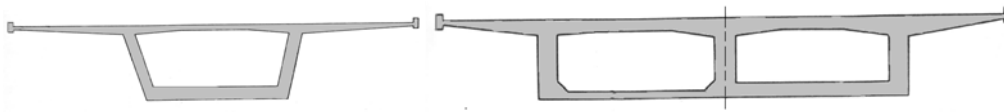


Figure 35 Cross-section of box-beam bridges [3]

Box bridges are more expensive to fabricate and can be more difficult to maintain due to the presence of bottom flange. Box bridges can be cast in situ with false-work or can be prefabricated and assembled on-site.

Scaffolding is a suitable choice for construction of short bridges. For bridges that are considerably long, the scaffolding needs to be moved between different sections of the bridge during construction. This is done by using travelling gantry technique. The construction method uses a movable supporting beam, gantry, on the false-work that reaches over at least one span but usually over the length of two spans. With special roller bearings and launching jacks the gantry can easily be moved forward along the bridge as the construction proceeds. The travelling gantry system is most suited for spans of 30 to 60 m (Figure 36).



Figure 36 Left) precast box-beam, Right) traveling gantry [4]

With even larger structures, incremental launching or balanced cantilever method can be used. Due to the fact that the cantilevers should balance each other out, it is important that only small segments are added on each side of the support (Figure 37). Usually in-situ segments are not larger



than 3 to 4 m or prefabricated segments between 1.8 and 3.5 m are used to limit the additional bending moments per step in order to avoid unbalance in the system.



Figure 37 Concrete box beam bridge erected with balanced cantilever method

Thanks to incremental launching it is possible to reduce the costs of construction of bridges longer than 150m and with spans between 30m-60m. One major advantage of incremental launching is that no scaffolding or false-work is used when the superstructure is assembled (Figure 38). The bridge can therefore pass without any problems over obstacles like rivers, buildings, railroads, etc. Another convenience is that the construction yard can be covered in order to enable a more protected environment against different weather conditions. Disadvantages of incremental launching are the additional costs for launching jacks, launching nose, additional pre-stressing, expenses for the construction yard and extra amount of concrete needed to increase the cross-section due to extra stresses caused by the launching.



Figure 38 Concrete box beam bridge during construction with incremental launching

3.5 Classification by static scheme

3.5.1 Beam bridges

Beam bridges are the simplest and the most commonly used type of bridges. The static scheme of such a structure can be single or multi span, simply supported beam and continuous beam (with or without joints) (Figure 39).

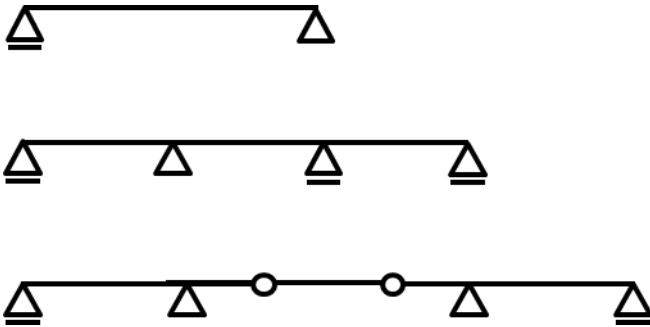


Figure 39 Different static scheme of the beam bridges: single span bridge, simply supported beam; multi-span bridge without joints; multi-span bridge with joints

3.5.2 Arch bridges

Main types of concrete arch bridges are closed spandrel arch, open spandrel arch and through arches (Figure 40). The cross-section of deck can be a slab, I-beam or box beam. For the closed spandrel arch, the span can be up to 80 m and for open spandrel arch is even up to 400 m.

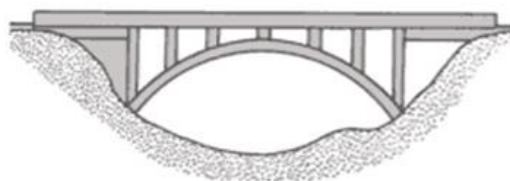
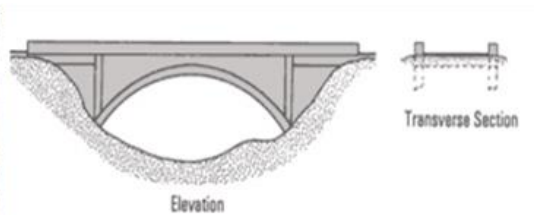


Figure 40 Main types of arch bridges; open and closed spandrel arch bridges



Open-spandrel arches are used for long and high crossings. Reinforced concrete arch bridges are ideal to span over river valleys or deep ravines. Although they are more complicated to design and construct, they have lower dead weight. The savings in material can be part of the cost-effectiveness, but the real savings comes because of the load-bearing of the arch itself, as well as the foundations upon which it rests, which could be made much less massive. However, the construction of the arch rib is very often the most crucial aspect that may influence the feasibility of the bridge. Available construction methods are casting concrete in situ on formwork and false-work supported directly on the valley or traveling formwork (Figure 41). Another method that is gaining popularity in recent years is to use precast concrete segments erected by the cantilever construction method with tiebacks.



Figure 41 Construction of arch bridge with traveling formwork method [5]

3.5.3 Frame bridges

Rigid frame bridges can either be single span or multi-span, Figure 42. Single span frame bridges are in general a slab beam and can span up to 15 m. The basic single span frame shape is most easily described as an inverted “U”. Multi-span frame bridges are used for spans over 15 m with slab or rectangular beam designs. Other common multi-span frame shapes is K-frame. Due to frame action between the horizontal and the vertical or inclined members, multi-span frames are not considered continuous. Concrete rigid frame are mainly casted in-situ.



Figure 42 Examples of the single and multi-span concrete frame bridges. On the left frame shape type “U” and on the right K-frame



3.5.4 Cable-stayed bridge

Cable-stayed bridges are often the most spectacular constructions. The reason for that is not only the aesthetic but also the possibility to achieve great span lengths. The span length for such bridges varies between 150 and 550 m, with the deck height 1/90-1/120 of the span, the height of pylons is between 0.16-0.22 of the bridge span. In general bridges with one pylon are used for the crossing of obstacles up to 300m. However pedestrian cable-stayed bridges with span of about 40m can also be built. Employing pre-stressed concrete, the superstructure can be made very slender using a few cable stays and a deck with a depth of only 25 to 30 cm. Figure 43 shows the longest reinforced concrete cable-stayed bridge in Poland with 4 spans: 50m – 256m – 256m – 50m, the height of the pylon is 122m.



Figure 43 Reinforced concrete cable-stayed bridge in Wroclaw in Poland constructed by Mostostal Warszawa S.A in 2011

The common construction method for multi-stay cable bridges is free cantilevering from the tower towards both sides (Figure 44). The final cables are used to support one segment after the other. Since full symmetry of loadings can usually not be secured, the tower must be stiffened under the deck by struts or by retaining cables from the tower head to suitable anchor points. For bridges with box girders, prefabricated segments with match cast joints can be used. Nonetheless using a paste in the joint will compensate for differential shortening during the curing and hardening period. Prefabricated segments can also be mounted, leaving a gap for overlapping longitudinal reinforcement by means of steel hinges on the webs, which allow adjustment for the correct positioning of the segment.



Figure 44 Erection of concrete cable-stayed bridge with cantilevering method

Three basic arrangements of the stay cables are shown in Figure 45.

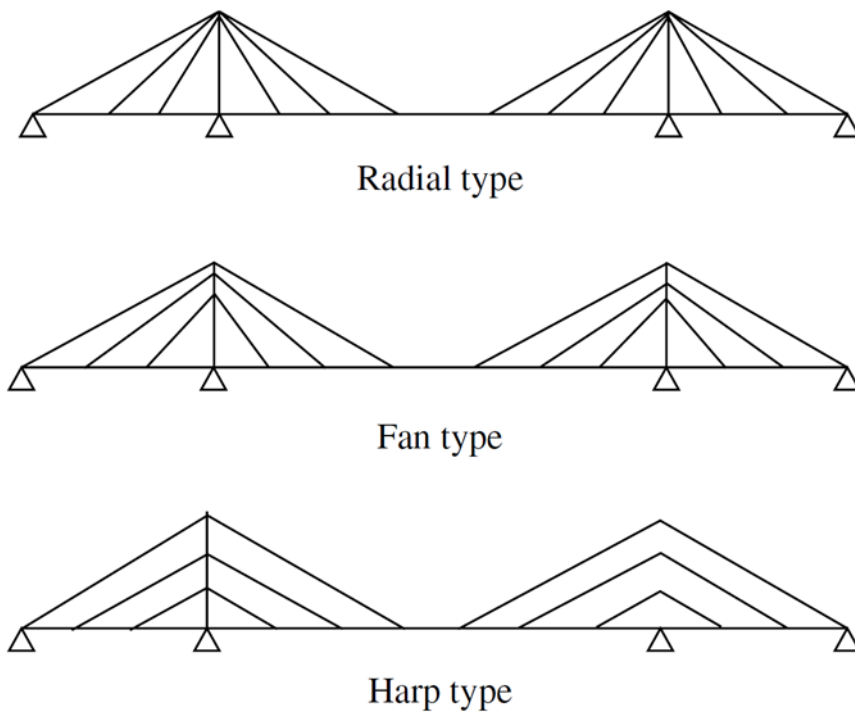


Figure 45 configurations of cable stayed bridges [7]



4 References

- [1] Reid Steel, Construction of Bridges, Available at <http://steel-bridges.com/advice.htm> (2011-10-04)
- [2] Madaj, A., Wołowicki W., 2003, Podstawy Projektowania Budowli mostowych, (Design of bridge infrastructure), Wydawnictwa komunikacji i Łączności, Warsaw, Poland.
- [3] Krupka, J., 2007, Stan techniczny mostów zintegrowanych na przykładzie wybranych obiektów w Polsce, <http://www.prz.rzeszow.pl/>
- [4] Kish, B., Langefors, P., 2005, Incremental launching versus scaffolding for construction of prestressed concrete bridges, Master's Thesis in the International Master's Programme Structural Engineering, Chalmers University of Technology, Sweden.
- [5] Zderic, Z., 2008, Equipment for erection of arch bridges, Chinese-Croatian Joint Colloquium Lon arch bridges.
- [6] Plewako, Z., Betonowe Konstrukcje Sprężone”, Politechnika Rzeszowska, Katedra Konstrukcji Budowlanych.
- [7] Chen, W., Lui, E., 2005, Handbook Of Structural Engineering, CRC Press.
- [8] GangaRad, H. V. S., Taly, N., Vijay, P. V., 2007, Reinforced Concrete Design With FRP Composites, CRC Press.
- [9] Ryall, M. J., Parke, G.A.R., Harding, J. E., 2000, Manual Of Bridge Engineering, Thomas Telford.
- [10] Strasky, J., 2005, Stress Ribbon and Cable Supported Pedestrian Bridges, Thomas Telford.
- [11] Troyano, L. F., 2003, Bridge Engineering: A global perspective, Thomas Telford.

D 2.1 – Appendix 2

Common problems in steel and concrete bridges





D 2.1 – Appendix 2

Common problems in steel and concrete bridges

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Colophon

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1 Common problems in steel bridges

This section presents the most common problems in superstructure of steel bridges which would be of interest when considering steel-concrete composite bridges in this project. Different problems have been categorized and examples and illustrations of each problem are presented.

1.1 Fabrication defects

Despite the rigorous specifications and the tight manufacturing tolerances to which structural components are rolled and formed, manufacturing and fabrication defects can and do find their way into completed structures. Rolling flaws may show up as delaminations, cracks, blisters, pits or inclusions as well as out-of-tolerance straightness. Such defects may be of little consequence, or they can help to initiate a future serviceability problem. Inferior welds and rough gas-cut edges can lead to major structural problems. A poorly formed or undercut weld, presence of slag inclusions or effects of frequent starts and stops could lead to an eventual fatigue problem. Unfortunately, few welding defects are observable, particularly once a structure is in service.

1.2 Corrosion

Corrosion is a common problem for steel and iron structures. Where the protective coating has not been maintained or an area of damaged coating not been repaired, corrosion takes place. 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 thus leads to reduction of stiffness of the structural member, see *Figure 1*. Decrease in the cross-sectional area also means increase in stresses in the member. The corrosion rate can be accelerated by the following factors:

- Presence of cracks and crevices;
- Different metals in contact (galvanic corrosion);
- Stray electrical currents;
- Ponding of moisture;
- Concentration of salts through evaporation;
- Chemical attack.



Figure 1 Loss of section, severe pitting due to condensation and concentration of salts on bottom flange (and lack of maintenance) [1]



Figure 2 Coating failure caused by inadequate primer [1]

1.2.1 Protective Coating Failure

It is rare for a protective coating to outlast the life of the structure. Breakdown of paint or loss of galvanizing is inevitable, and should be anticipated. The rate of breakdown depends on a number of inter-related factors, with “time of wetness” being the most important. This usually results from condensation and may be increased by absorption of moisture by wind-borne salts on areas not subjected to rain washing. Accumulation of debris, bird droppings, flaking paint etc. will all retain moisture and promote corrosion.

In addition to eventual failure of a coating system by weathering, premature failure may result from:

- Loss of coating adhesion due to faulty specification or application;
- Incompatibility of successive coats;
- Subsurface rusting due to inadequate surface preparation and/or priming paint;
- Localized failure due to mechanical damage;
- Inadequate film-build on sharp edges welds and paint “shadow areas”.

In some cases, specialist advice may be required to establish the cause and recommend suitable remedial work. Figure 2 to Figure 6 illustrate some typical examples of problems with corrosion protection later in steel bridges.



Figure 3 Incompatible coatings (alkyd paint over zinc-rich) [1]



Figure 4 Loss of adhesion (inadequate surface preparation) [1]



Figure 5 Damp patch caused by accumulated rust and debris [1]



Figure 6 Effect of rain washing: removal of marine salts by rain washing has kept bottom of outer beam corrosion-free for 14 years (red lead/MIO alkyd system) [1]

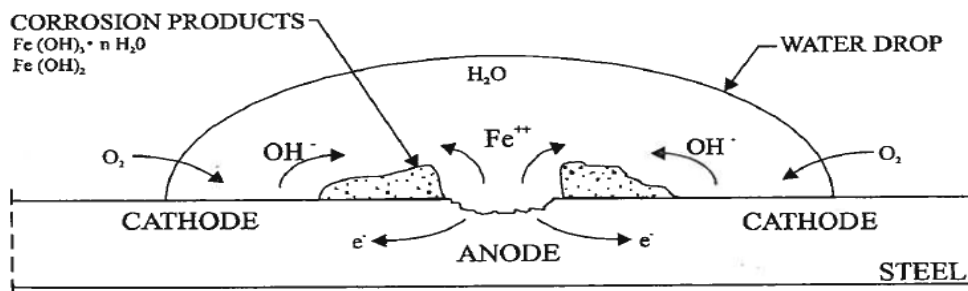


1.2.2 Types of corrosion

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 7[2].



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

Figure 7 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 [2,3].

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 leading to tear forces causing damage in the material, see Figure 8.

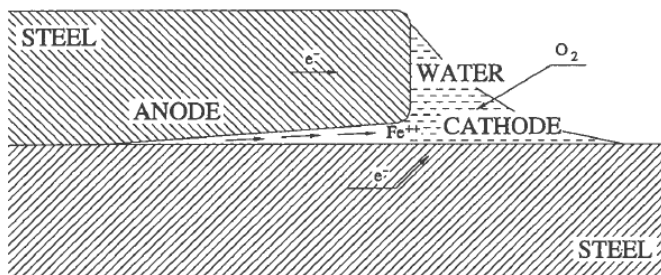




Figure 8 Process of crevice corrosion [2]

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

Some of the elements which should be present in order for galvanic corrosion to occur are [3]:

- 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 ratio of the anode and cathode areas. 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. Stress corrosion is very common in cables in cable stayed as well as suspension bridges. In structural elements stress corrosion is not very common [3]. Some details prone to stress corrosion are given below [3]:

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

1.3 Impact damage

Impact damage caused by vehicles can result in distortion, tearing or cracking in bridge components, see Figure 9. Sometime damage in bridge components do not need to be repaired and left unrepaired depending on how the damage influences the load carrying capacity. The examination of the damage should be done accurately.



Figure 9 Accident damage in a steel girder caused by an over-height vehicle [4]



1.4 Fatigue

Fatigue is known as the process of progressive localized mechanical damage occurring in a structural element, whose material is subjected to fluctuating stresses and strains at some point or points. Accumulated damage may result in loss of stiffness and development of micro- and macro-cracks, eventually leading to complete failure after a sufficient number of fluctuations.

In most cases, fatigue cracks initiate in regions that are subjected to high local stresses often in combination with local defects in the material. Examples at which high stresses might exist are holes, welds, abrupt changes of shape, cracks or other defects. On the contrary, in adjacent parts where the stress state is insignificantly lower, no crack would initiate. Hence, fatigue is clearly a localized process where the crack originates in a location where several micro-cracks are formed under repeated loading. These micro-cracks then grow together into one dominant crack.

The mentioned process in which a crack is formed after a coalescence of several micro-cracks is called the initiation phase of fatigue crack growth and is a result of plastic deformations in a small area in front of the crack tip. The presence of plastic deformations also implies that, fatigue is an irreversible process which leaves permanent structural damage.

Common reasons for fatigue damage are classified as:

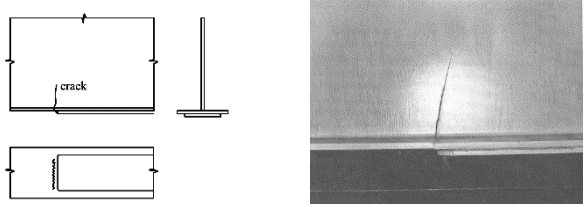
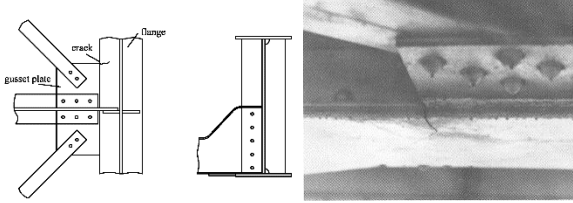
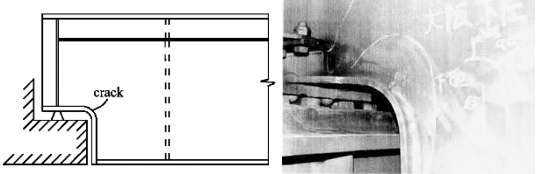
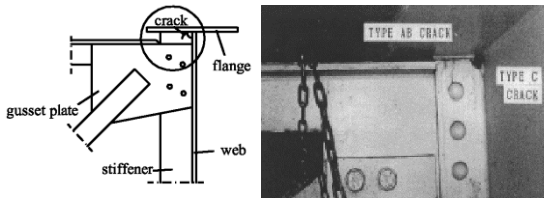
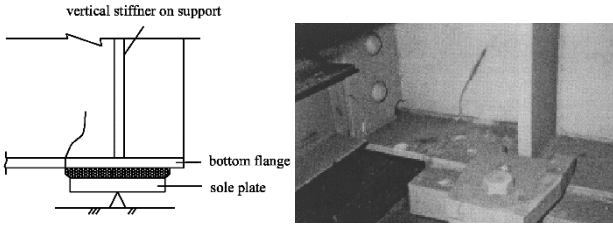
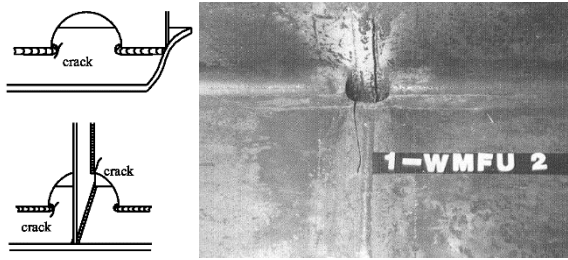
- (1) Welding defects included during fabrication
- (2) Inappropriate structural details of low fatigue strength
- (3) Stresses and deformations unforeseen in design of joints
- (4) Unexpected effects such as vibrations, etc.

Older structures require special consideration with regard to fatigue since often it was not taken into account in the design. Avoiding or reducing the effect of fatigue can be done, for example, by reducing the amount of load and loading cycles, increasing the cross sectional area of the member, reducing the load on the member by redistributing it onto additional members. Table 1 shows an overview of typical fatigue failures of welded structures.

Table 1 Typical fatigue failures of welded structures and remedial measures often used in practice [5]

Fatigue failure	Example
<p>Transverse groove welds Cracks in the butt connection groove weld of tension-side longitudinal stiffener or flanges</p>	



Fatigue failure	Example
<p>Cover end plates Cracks at transverse front welds at cover plate end</p>	
<p>Gusset plates on flanges Cracks in welded gusset plate joint on flanges</p>	
<p>Coped end of deck plate girder Cracks at the coped end of deck plate girders</p>	
<p>Cross bracing connections Fatigue cracks at the cross bracing connection detail on the upper flange</p>	
<p>Sole plate connection Fatigue crack in sole plate connection detail at support</p>	
<p>Cut out web Fatigue crack in the web or flange, initiated at the fillet weld toe of the cut out web</p>	



Fatigue failure	Example
<p>Orthotropic deck</p> <p>Orthotropic steel bridge deck, different details with low fatigue strength</p>	
<p>Hangar and pinned connections</p> <p>Cracks induced by vibration, e.g. by wind or traffic</p>	
<p>Transverse stiffener web gaps</p> <p>Cracks near the end of a vertical web stiffener</p>	
<p>Floor-beam bottom flange</p> <p>Fatigue crack occurred in the coped web of the end floor-beam</p>	

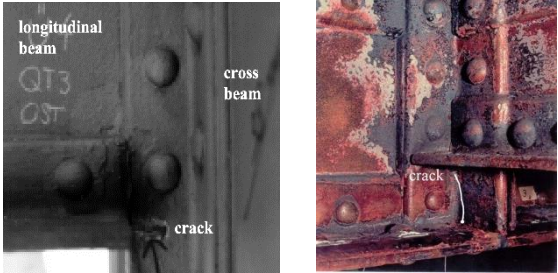
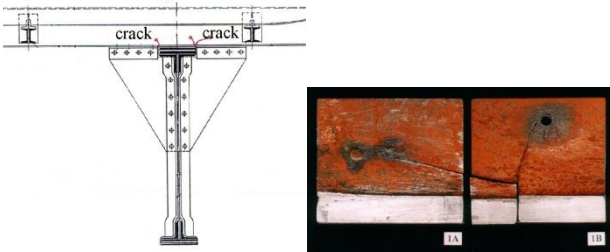
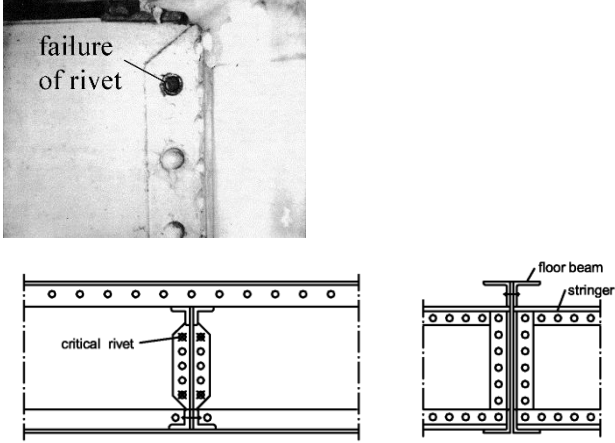
Table 2 shows an overview of typical fatigue failures in riveted and bolted structures.



Table 2 Typical fatigue failures of riveted and bolted structures and remedial measures often used in practice [5]

Fatigue failure	Example
<p>Cover plate end</p> <p>Cracks initiating at the holes in net cross section at the end of cover plates due to overload and due to changes of geometry</p>	
<p>Cover plate end</p> <p>Cracks in the flange of the gross cross section due to changes of geometry</p>	
<p>Gusset plates</p> <p>Cracks in gusset plates due to insufficient thickness</p>	
<p>Ballast sheets</p> <p>Cracks in barrel shaped ballast sheets</p>	



Fatigue failure	Example
<p>Cross beam – longitudinal beam connection</p> <p>Cracks in a connection between cross beam and longitudinal roadway beam</p>	
<p>Bearings</p> <p>Cracks due to frozen bearings or joints, e.g. because of corrosion or temperature differences</p>	
<p>Stringer to floor-beam connection</p> <p>Fatigue failure of rivet heads due to localised due to local bending of the rivet head</p>	

1.5 Brittle fracture

The term brittle fracture refers to failure of steel material at very low strains or basically failure without displaying ductility. Usually the main reason of brittle fracture in steel is very low service temperature. However, changes in the micro-structure of the material, for example by welding, might result in brittle fracture too.

Brittle fracture results in unpredicted loss of strength and endangers the safety of the structure. This type of failure is the most critical when the structure has no signs of fatigue cracking. Brittle failure usually takes place without any warning and at stress level below the ultimate strength of the material.



1.6 Loose or defective fasteners

Whether operating in shear or in a friction grip joint, fasteners must be properly installed to function properly. Sometimes, because of excessive vibration, over-straining, corrosion or improper installation, fasteners can become loose and should be replaced. Specific problems typically associated with various types of fasteners are [1]:

- Rivets can become loose and can also suffer from loss of head section due to corrosion if the protective coating is not maintained;
- Mild steel bolts tend to corrode rapidly if the protective coating is not intact. This type of bolt may also loosen with vibration unless suitable washers or lock nuts are provided;
- High-strength bolts will also corrode unless the protective coating is maintained. Galvanized bolts are usually better than painted 'black' steel. Improperly torqued bolts will loosen and bolts which have been installed through heavily tapered flanges without suitably tapered washers may bend and become overstressed;
- 'Huck' fasteners might not be installed to the manufacturer's specifications. The collar must be correctly swaged onto the pin which must be of the correct length for the particular joint. Improperly installed fasteners are unlikely to provide the correct clamping force across the joint. Even when using galvanized fasteners, the collar needs a full protective coating to prevent corrosion;
- Nuts might be of a material incompatible with the bolts or the material being joined. This may lead to electrolytic action if not separated by a non-conductive washer;
- Load indicating washers might be incorrectly installed. The gap provided by the protrusions can be outside the manufacturer's tolerances;
- Spring washers can corrode and/or fracture.

1.7 Joints

Bridges are under movement during their lifetime. The movements could be due to temperature and humidity changes; creep, shrinkage and cyclic effects; axial and flexural strains arising from dead and live loads and pre-stressing; dynamic load effects, including vehicle braking and lurching, centrifugal forces; and long-term movements, such as those caused by settlement and earth pressure. For a bridge to function as intended, it must be capable of accommodating all such movements. Movement is usually accommodated by bridge bearings and deck expansion joint systems. A properly functioning bridge deck joint accommodates the horizontal and vertical movements of the structure while providing smooth ride ability, low noise level and wear resistance.

Damage to expansion joint might be categorized as following:



- Deterioration of joint elements
- Loss of sealant and leakage
- Spalling of the deck
- Accumulation of debris in the joint

Figure 10 to Figure 13 present some common problems observed in expansion joints.



Figure 10 Joint in poor condition [6]



Figure 11 Joint in poor condition [6]



Figure 12 Seal has failed leading to extensive water leakage through the joint [6]



Figure 13 Loose and broken protection angle [6]

1.8 Railings

One of the most important safety features of bridges is the railing. The primary function of a bridge railing is to keep errant vehicles from driving off the bridge. Bridge railing must smoothly direct the vehicles in such a manner that they do not overturn and consequently fail. Railing system includes rail posts or supports and railings. The railing posts are connected to the edge beam or the rim of the bridge deck by bolts or should be casted into or form part of the concrete slab or edge beam.

Railing systems can suffer from the following problems:

- Impact damage



- Corrosion
- Defects in connections

Figure 14 to Figure 17 show some typical damage in railing systems.



Figure 14 All the nuts and parts of bolts completely corroded off [6]



Figure 15 Badly damaged metal railing with missing post and torn guard fence rail [6]



Figure 16 Rail connector missing [6]



Figure 17 Weld crack at the base and corrosion of nut [6]

1.9 Bearings

Bridge bearings are devices for transferring loads and movements from the deck to the substructure and foundations. They are classified into elastomeric bearings and metal bearings.

Damage to elastomeric bearings includes:

- Deterioration
- Excessive shear deformation
- Misalignment
- Dowels (missing or distorted dowels)

Damage to metal bearings could be categorized into:

- Deterioration
- Problems with protective coating system
- Horizontal and vertical misalignment



- Problems with bearing support element
- Problems with lubricating system
- Problems with debris and contamination

Figure 18 Figure 21 show some examples of defects in bearings.



Figure 18 Significant splitting and bulging of bearing pads [6]



Figure 19 Significant bulging of bearing pad [6]



Figure 20 Significant corrosion with pitting [6]



Figure 21 Significant loss of support under the bearing [6]

1.10 Faults due to poor detailing

Defects can occur in a bridge structure due to poor design, detailing and specification. This category can include:

- The abrupt curtailment of steel section flanges in tension members;
- Excessive eccentricities (both in plane and out of plane) in joint intersections;
- Inadequate provision for rotation;
- Poor drainage provisions;
- Curtailment of welds in inappropriate locations.



1.11 External causes of damage

Environment, wildlife, fire and vandalism are typical examples of external causes for damage. 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 [3]. The acetic acid from deck or members in structures made of timber causes damage of the protection coatings and 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 flanges [2].

Fire can be caused by vandalism or vehicle impact and result in weakening of the structure. Degree of the deterioration should be analyzed and examined and repair measures should be applied if required [2].



2 Common problems in concrete bridges

In this chapter, the most common problems that may occur in concrete bridges during their service life are described.

2.1 Symptoms of distress and deterioration of concrete

2.1.1 Scaling

Scaling is local flaking or peeling of a finished surface of hardened concrete as a result of exposure to freezing and thawing, see Figure 22. Generally, it starts as localized small patches which later may merge and extend to expose large areas. Light scaling does not expose the coarse aggregate. Moderate scaling exposes the aggregate and may involve loss of up to 3 to 10 mm of surface mortar.



Figure 22 Example of scaling in concrete

In severe scaling more surface area would be lost and the aggregates are clearly exposed and stand out. Concrete exposed to freeze and thaw in the presence of moisture and/or de-icing salts are susceptible to scaling. Most common reasons for scaling include:

- The use of non-air entrained concrete or too little entrained air. Adequate air entrainment is required for protection against freezing and thawing damage. However, even air-entrained concrete will scale if other precautions are not considered.
- Application of excessive amounts of calcium or sodium chloride de-icing salts on concrete with inadequate strength, air entrainment, or curing. Chemicals such as ammonium sulphate or ammonium nitrate, which are components of most fertilizers, can cause scaling as well as induce severe chemical attack on the concrete surface.
- Any finishing operation performed while bleed water is on the surface. If bleed water is worked back into the top surface of the slab, a high water-cement ratio and, therefore, a low-strength surface layer is produced. Overworking the surface during finishing will reduce the air content in the surface layer, making it susceptible to scaling in freezing conditions.



- Insufficient curing. This omission often results in a weak surface skin, which will scale if it is exposed to freezing and thawing in the presence of moisture and de-icing salt.

2.1.2 Spalls

Spalls are the loss of concrete from a concrete member, see Figure 23. They can be provoked by the low quality of concrete additives, improper concrete mixture, corrosion of reinforcement in concrete, ineffective drainage system or freeze and thaw cycles.



Figure 23 Example of spalls in concrete

Spalls may range in extent from being minor to severe according to Table 3.

Table 3 Range of the concrete spalls

	Small spalls	Medium spalls	Large spalls	Very large spalls
Depth	Less than 5mm	5-10 mm	10-25mm	More than 25 mm

With minor spalls the aggregate within the concrete is exposed, whereas moderate spalls are deeper. In such as case, for example, the pre-stressing strands or reinforcing steel is exposed. With severe damage, spalls will be deep and the inner structure of the girder is not only exposed but may also be damaged. Size of the spalls is strongly related to:

- Thickness of the concrete cover,
- Stiffness of the bridge,
- Traffic flow,
- Corrosion intensity and damages,
- Number of thaw and freezing cycles,
- Concrete porosity.

Spalling is more prevalent in light-weight concrete and the aggregate exposed is more porous, which can lead to freeze/thaw damage that would not occur in normal concrete. Spalls can appear in the first year of the service of the structure or after few years.

2.1.3 Cracks



The damage of concrete bridges is characterized by various types of cracks due to the immanent nature of concrete as a brittle material. The number and depth of cracks are critical factors to specify the damage.

In order to evaluate the impact and consequence of cracks on bridge safety and durability, the causes that lead to the cracks in concrete structures are important to be identified. In general the cracks in concrete bridges include:

- (1) Settlement or subsidence cracks. These cracks are formed over and parallel to the top-most reinforcement as the concrete settles around the bars as it dries.
- (2) Plastic shrinkage cracks. These cracks appear in the deck when the evaporation rate exceeds the bleed rate of newly placed plastic concrete. Extreme environmental conditions and high concrete temperatures increase the surface evaporation rate, and thus render the deck vulnerable to plastic shrinkage cracks, see Figure 24.
- (3) Dry shrinkage cracks. Concrete is usually mixed with more water than is needed to adequately hydrate the cement. The remaining water, known as water of convenience, evaporates, causing the concrete to shrink. Restraint to shrinkage, provided by the sub-grade, reinforcement, or another part of the structure causes tensile stresses to develop in the hardened concrete and eventually cracking in the concrete, see Figure 25. In many situations, drying shrinkage cracking is inevitable. Therefore, contraction (control) joints are routinely placed in concrete to predetermine the location of drying shrinkage cracks
- (4) Temperature introduced cracks due to variation of temperature in large concrete mass. Unrestrained concrete undergoes volumetric changes as it experiences temperature variations. The mechanism that causes thermal cracks in decks is very similar to that which causes drying shrinkage cracks.
- (5) Flexural cracks. They can occur when the concrete is in its initial maturing stage after placement as well as in service, see Figure 26. Flexural cracks can develop in the negative moment regions of the concrete over the interior supports of continuous spans due to the dead weight of the girders plus the newly placed concrete. When the deck is in service, the addition of live loads can also cause cracking in the negative moment regions. Furthermore, gravity loading can cause under-deck cracking in the positive moment regions of both simply supported and continuous spans
- (6) Shear cracks. High shear stresses can cause cracks in a concrete member, see Figure 27. Cracks are formed when the principal tensile stress in the concrete reaches a critical value, for instance, the concrete tensile strength. The crack will form normal to the direction of the principal tensile stress. The inclination and the magnitude of the principal tensile stress depend on the total stress state.



Cracks that appear in not “natural” locations in concrete structures or have large spread may show that the structure is overloaded and indicate emergency state. Relatively common in reinforced concrete structures are small cracks perpendicular to the beam axis, that are located in the areas with highest bending moments. Diagonal cracks in reinforced concrete and all the cracks in pre-stressed concrete (except partially pre-stressed structures, in which perpendicular cracks to the beam axis provoked by temporary loads are allowed) can be extremely dangerous. In general, diagonal cracks around support areas indicate high shear stress. Failure in shear is not a ductile failure mechanism and thus can take place without warning. Permanent cracks that occur in pre-stressed concrete may provoke serious risks for the structure and its users. The cracks may be caused by too low pre-stressing force. This is related often to not good enough design and construction process or loss of pre-stressing related to rheological effects.

Cracks that are parallel to the beam axis are in general caused by corrosion of reinforcement, although they can be also be related to the shrinkage process. Sometimes such cracks are caused by lack of adhesion between steel and concrete.

Cracks that can be observed (usually one crack across the bridge) in the bridge span can be caused by improper location of reinforcement. Such a condition contributes to the significant decrease of bearing capacity.

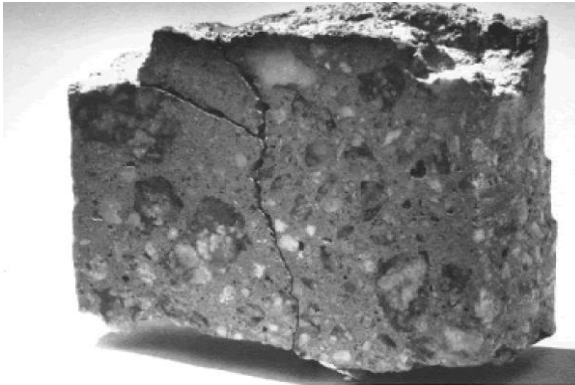


Figure 24 Example of plastic shrinkage cracks [7]



Figure 25 Example of drying shrinkage cracks [7]



Figure 26 Example of flexural cracks [7]

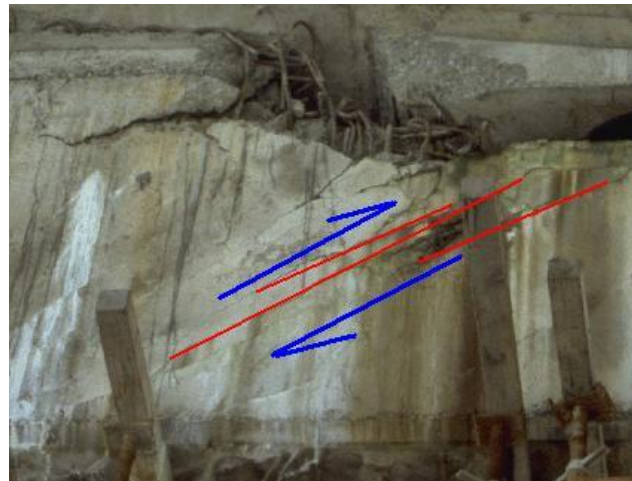


Figure 27 Diagonal shear cracks over support area

2.1.4 Delamination

Concrete delamination is the separation of the top surface from the underlying concrete; it is analogous to a skin blister that usually occurs at or just above the top reinforcement in the deck, Figure 28. Concrete delamination occurs when the top surface of the concrete is densified or sets up before the water in the concrete mixture (the bleed water) and air have a chance to reach the surface. This densified layer is usually thin but it will not allow the air and bleed water to pass to the surface. The dense surface interferes with upward motion of water that results from the settling of solids within the concrete mixture. As the water meets the densified thin layer, it migrates laterally separating the surface from the body of concrete. The dense top film can form when the surface is prematurely troweled. The problem sometimes occurs when the environmental factors rapidly dry the surface of the concrete. This can include: wind, sun, low humidity, or drying the surface with heaters. This creates conditions making it appear that surface is ready for finishing while the underlying concrete is still plastic and is bleeding air and water. Vapor barriers beneath concrete force all of the bleed water to rise compounding the potential for delamination.

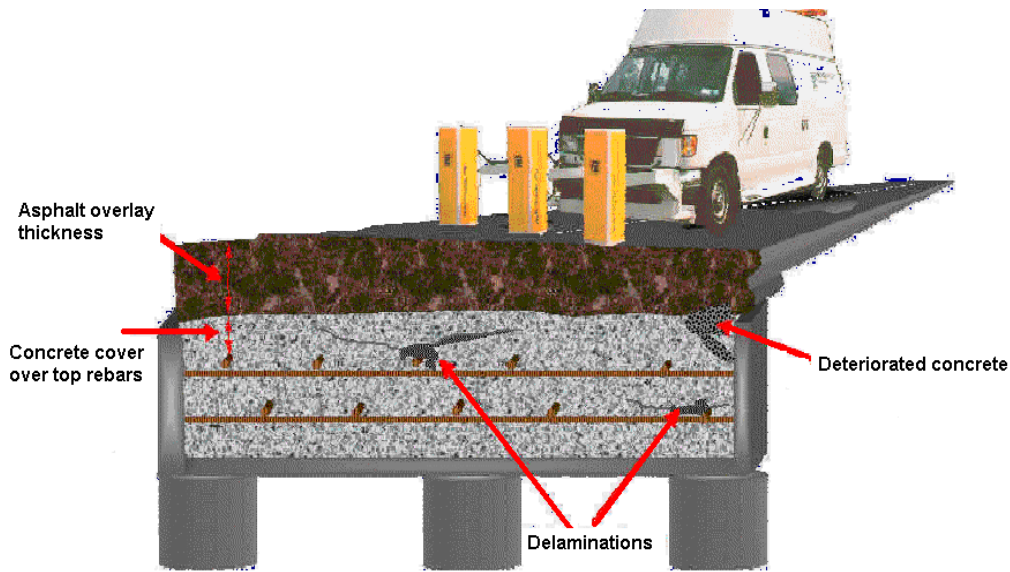


Figure 28 Scheme of delamination of concrete deck [7]

Any factor that delays the set of the underlying concrete or promotes rapid surface drying increases the potential for delamination. The National Ready Mix Concrete Association (NCRMA) states the following conditions increase the probability of concrete delamination:

- Slow curing of underlying concrete caused by cool or cold substrates. This condition occurs more frequently in the spring and autumn when soils are cool and the daytime temperature is rising
- The addition of concrete retarders or cold water to a mix to retard curing
- High air entrainment levels or air contents higher than necessary
- Environmental factors causing rapid surface drying making the surface "crust" and appear ready to finish
- Excessive consolidation of concrete that brings too much mortar to the surface
- Use of dry shake materials on concrete, particularly with air entrained concrete
- Placement of thick slabs
- Placing concrete directly on a vapor barrier
- Corrosion of reinforcing steel in concrete,
- Cyclic freezing.



Figure 29 Delamination provoked by the corrosion of reinforcement [7]

2.2 Causes of distress and deterioration of concrete

2.2.1 Corrosion

Corrosion of reinforcing bars and pre-stressing tendons is one of the most significant and unremitting factors in the process of deterioration of bridges. In combination with water and oxygen, the main ingredient for corrosion is chloride ions from applications of de-icing salts or marine exposure of concrete bridge members, Figure 30. Chloride ions eventually penetrate the concrete cover; react with embedded reinforcement to form expansive corrosion products, causing concrete to crack with eventual concrete spalling due to debonding of concrete accelerated by traffic induced vibrations.

The corrosion is mainly provoked by aggressive external factors or/and chemical reactions in concrete. Concrete resistance is strongly related to mechanical actions i.e. cracks, abrasion, etc. Those mechanical defects increase penetration of aggressive environment to the internal layers of concrete. The consequences are also an acceleration of concrete deterioration caused by chemical and physical corrosion.



Figure 30 Corrosion damage to concrete bridge pier [9]

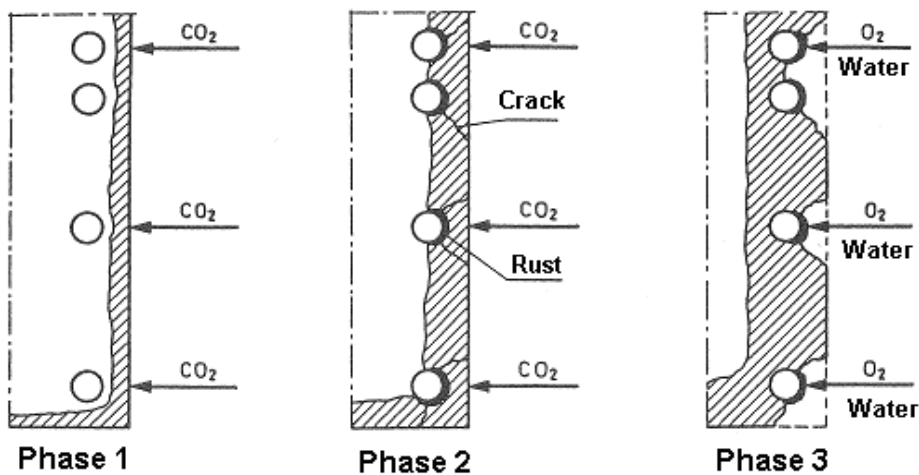


Figure 31 Phases of concrete deterioration [10]

Corrosion caused by aggressive external factors provokes the change of the chemical formula. This contributes to the concrete deterioration; this process usually starts from the concrete surface and progresses gradually inside the concrete core. Internal corrosion is caused by chemical reactions of hardened cement binder and aggregate. This type of corrosion may be the effect of using cements with high content of sulphur, aggregate contaminated with the acidulated batched water.

Corrosion gradually deteriorates concrete properties and leads to its damage. Scope of the damage and its propagation depends on the aggressive factors and mechanical actions, concrete properties (including tightness), resistance of cement binder and aggregates. Cement binder is the most vulnerable concrete additive. Susceptibility of the binder is related to high reactivity of the calcium hydroxide, hydrated in the cement hydration process. Calcium hydroxide is a base that gives the



elements $\text{pH}=12-13,5$. Indicator of the low quality of concrete environment is $\text{pH } 6,5$, such concrete is not able to protect steel reinforcement from the corrosion.

For post-tensioned concrete bridge members (both external and internal pre-stressing), the voids in grouted ducts and/or excessive bleed water (in certain grout mix designs), in addition to chloride/water entering the rough breached ducts or faulty joints at anchorage locations, will corrode the uncoated strands, see Figure 32. The difficulty is that there are no reliable, rapid and cost-effective non-destructive methods to assure owners that completed post-tensioned concrete structures have met the construction specifications. One of the major inspection concerns is whether the ducts in post-tensioned concrete members have been completely filled with grout and whether there is a uniform coverage over the pre-stressing steel.



Figure 32 Corroded Strands [9]

In many cases the ducts have large voided sections and are only partially filled with grout. In addition, it is very difficult to assess the condition of anchorage areas. Past research has identified that excessive bleed water in certain commercial grouts corroded the strands in a very short time, and that in due time under load, these corroded strands can break prematurely. Since post-tensioned concrete members rely on the tensile strength of the strand to resist loads, loss of a few strands in members could be catastrophic. Even small pits lead to fracture of a strand, as compared to reinforced concrete where the reinforcing will literally rust away (if preventative measures are not taken) without breaking.

2.2.2 Alkali Aggregate Reaction (AAR)

Alkali Aggregate Reaction (AAR) is a chemical reaction of alkali in concrete and certain alkaline reactive minerals in aggregate produce a hygroscopic gel which, when moisture present, absorbs water and expand. Gel expansion causes cracking in the concrete, see Figure 33. The number of structures affected by AAR is relatively small comparing to the total number of concrete structures built, but the problem has been found in many countries around the world.

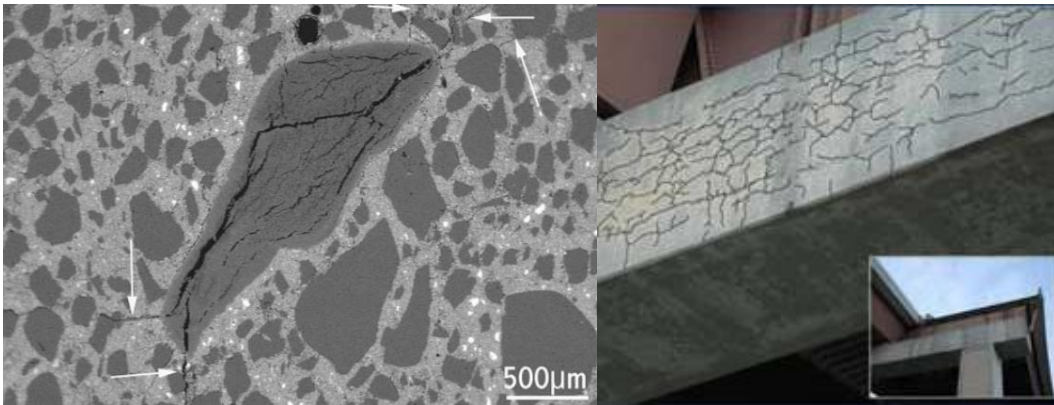


Figure 33 Example of the cracks in concrete due to the Alkali Aggregate Reaction [8]

Most of the structures severely cracked by AAR are exposed to weathering or in contact with damp soil. This is because for significant expansion to occur, sufficient presence of moisture is essential. Apart from the moisture, high content of alkali in the concrete is also essential. No case has been found where the alkali content, in terms of equivalent sodium oxide ($\text{Na}_2\text{O}_{\text{equi}}$), is below 3-4 kg per cubic meter of concrete.

It is also found that, when there is sufficient moisture and alkali, maximum expansion of concrete due to AAR occurs when the content of reactive minerals in aggregate is within a sensitive region. Content of reactive minerals below or greater than the worst value, AAR expansion reduces. For AAR expansion damage to occur, it is necessary to have: sufficient moisture supply, high content of alkali in concrete or reactive minerals in aggregate in “sensitive” regions.

There can be several sources of alkalis in concrete:

- **Cement:** All ingredients of concrete may contribute to the total alkali content of the concrete; the major source of alkali is from cement. The chemical composition of cement is usually expressed in terms of oxides. In relation to AAR, alkali content in cement is determined from Na_2O and K_2O .
- **Pozzolans:** A pozzolan is a siliceous or siliceous and aluminous material which reacts with lime released from cement hydration forming a compound possessing cementitious properties. Pozzolanic materials are used as a cement replacement or as part of cementitious material to modify or improve properties of concrete, sometimes for economical consideration. Common pozzolanic material used in concrete include PFA (pulverized fuel ash, or fly ash), silica fume, GGBS (ground granulated blast-furnace slag). Other pozzolans include volcanic ash (the original pozzolan), opaline shale and chert, etc. Pozzolan consumes alkali when react with lime. When considering pozzolan contribution of alkali to concrete, a reduction to the alkali content of the pozzolan should be allowed for.



- **Aggregate:** Aggregate containing feldspars, some micas, glassy rock and glass may release alkali in concrete. Sea dredged sand, if not properly washed, may contain sodium chloride which can contribute significant alkali to concrete.
- **Admixtures:** Admixture in the context of AAR in concrete means chemical agents added to concrete at the mixing stage. These include accelerators, water reducers (plasticizers), retarders, super plasticizers, air entraining, etc. Some of the chemicals contain sodium and potassium compounds which may contribute to the alkali content of concrete.
- **Water:** Water may contain certain amount of alkali.
- **Alkalis from outside the concrete:** In areas with cold weather, de-icing salt containing sodium compounds may increase alkali content on the surface layer of concrete. Soils containing alkali may also increase alkali content on the surface of concrete.

2.2.3 Abrasion damage and mortar flaking

Abrasion damage in wheel tracks can be caused by studded tires and chain wear, see Figure 34. Such damage can also be caused by the blades of snow ploughs. In addition; abrasion damage manifests itself as polishing of the aggregates which can lead to a slippery surface.



Figure 34 Example of abrasion damage [7]



Figure 35 Example of mortar flaking damage [7]

Mortar flaking is a loss of surface mortar from above the top side of near-surface aggregate particles, see Figure 35. It is often caused by prolonged finishing of thin surface mortar over the top side of near-surface aggregate particles and inadequate curing.

2.2.4 Freeze

Deterioration of concrete from freeze thaw actions may occur when the concrete is critically saturated, which is when approximately 91% of its pores are filled with water. When water freezes



to ice it occupies 9% more volume than that of water. If there is no space for this volume expansion in a porous, water containing material like concrete, freezing may cause distress in the concrete. Distress to critically saturated concrete from freezing and thawing will commence with the first freeze-thaw cycle and will continue throughout successive winter seasons resulting in repeated loss of concrete surface. Effects of the freezing –thawing process can be observed in Figure 36.

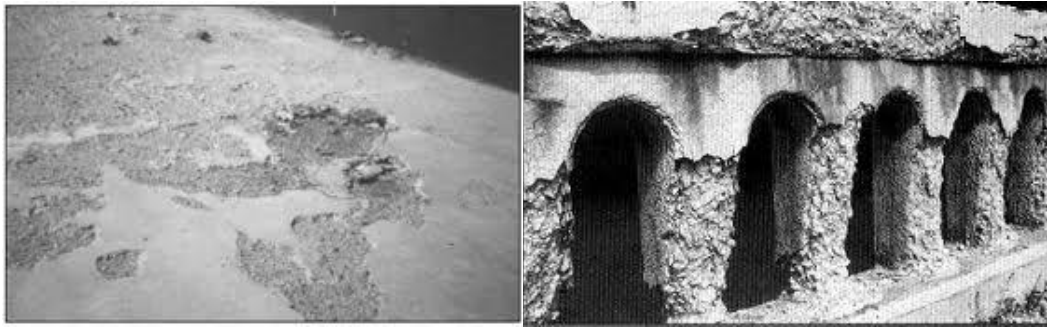


Figure 36 Example of concrete deterioration caused by freezing-thawing [7]

To protect concrete from freeze/thaw damage, it should be air-entrained by adding a surface active agent to the concrete mixture. This creates a large number of closely spaced, small air bubbles in the hardened concrete. The air bubbles relieve the pressure build-up caused by ice formation by acting as expansion chambers. About 4% air by volume is needed and the air-bubbles should be well distributed and have a distance between each other of less than 0.25 mm in the cement paste. Concrete with high water content and high water to cement ratio is less frost resistant than concrete with lower water content.

Deterioration of concrete by freeze thaw actions may be difficult to diagnose as other types of deterioration mechanisms such as Alkali Silica Reactions (ASR) often go hand in hand with freezing-thawing. Frequently, it is maybe difficult to evaluate which mechanism caused the initial damage, however, if all other mechanisms can be excluded the typical signs of freezing-thawing are:

- Spalling and scaling of the surface,
- Large chunks i.e. cm size,
- Exposing of aggregate,
- Usually exposed aggregate are un-cracked,
- Surface parallel cracking,
- Gaps around aggregate.

2.2.5 Sulphate attack

Sulphate attack is a chemical breakdown mechanism where sulphate ions attack components of the cement paste. The compounds responsible for sulphate attack are water-soluble sulphate-containing



salts, such as alkali-earth (calcium, magnesium) and alkali (sodium, potassium) sulphates that are capable of chemically reacting with components of concrete. When sulphate gets into concrete, it combines with the C-S-H, or concrete paste, and begins to destroy the paste that holds the concrete together. As sulphate dries, new compounds are formed, often called ettringite. These new crystals occupy empty space, and as they continue to form, they cause the paste to crack, further damaging the concrete.

There are two kinds of sulphate sources: internal and external. Internal source is rarer but, originates from such concrete-making materials as hydraulic cements, fly ash, aggregate, and admixtures. Internal sulphate attack can be related to:

- Over-sulphated Portland cement,
- Presence of natural gypsum in the aggregate,
- Admixtures also can contain small amounts of sulphates.

External sources of sulphate are more common and usually are a result of high-sulphate soils and ground waters, or can be the result of atmospheric or industrial water pollution. External sulphate attack may be related to:

- Soil that contains excessive amounts of gypsum or other sulphate,
- Ground water be transported to the concrete foundations, retaining walls, and other underground structures,
- Industrial waste waters.

Sulphate attack processes decrease the durability of concrete by changing the chemical nature of the cement paste, and of the mechanical properties of the concrete.



Figure 37 Spalling caused by sulphate attack [11]

The microscopic appearance of concrete suffering from external sulphate attack appears to be quite variable. Some diagnostic features such as

- Surface parallel cracks,
- Presence of gypsum and ettringite,



- Depletion of calcium hydroxide,
- Decalcification of C-S-H

are, however, often found associated with external sulphate attack. The above-mentioned features are usually most pronounced near the attacked surface. Sometimes external sulphate attack causes the paste to expand and gaps around aggregates are formed. All the features do not necessarily have to be observed to diagnose an external sulphate attack. The features present depend on many factors such as the quality of the concrete (including w/c and cement type), exposure time, temperature, concentration, and chemistry of the ambient water. There is a general agreement that concrete suffering from external sulphate attack develops a more and less pronounced mineralogical and chemical zoning which can be studied in the optical fluorescence microscope and the scanning electron microscope.

2.2.6 Carbonation of concrete

Atmospheric carbon dioxide can react with most cement hydrates. This reaction may very slowly move into unsealed concrete pores, cancel alkalinity and set up a steel corrosion process [13]. Such process is called carbonation of concrete and causes reduction of pH of concrete (pH of the fresh cement paste is at least 12.5 and about 7 for a fully carbonated paste). The speed of carbonation is affected by the following factors [10]:

- structure (porosity),
- water content in capillaries,
- the partial pressure of CO₂,
- Ca(OH)₂ content in concrete.

Pores and cracks in the concrete, cause a larger surface area in contact with air and speed up carbonation. The process of carbonation requires the presence of H₂O because CO₂ dissolves in water forming H₂CO₃. However, when the concrete is completely wet, CO₂ does not have access to the concrete and the process is stopped. The process is progressing quickly for relative humidity between 40 to 90%. Also the shape of the elements affect the speed e.g. convex corners are better penetrated by CO₂ than concave, so usually there are most carbonated, see Figure 38.



Figure 38 The effects of carbonation on concrete (photo by MADe)

Carbonation process increases the strength of the concrete compressive strength. Therefore, it is desirable for the construction made of unreinforced concrete or reinforced concrete with non-metallic rods (e.g. FRP materials).



3 References

- [1] <http://www.nzta.govt.nz/resources/bridge-inspection-maintenance-manual/docs/4-structural-steel.pdf>
- [2] Radomski, W., 2002, Bridge Rehabilitation. Imperial College Press, ISBN 1860941222.
- [3] Tilly, G. P., Matthews, S. J., Deacon, D., De Voy, J. , Jackson, P.A., 2008, Iron and steel bridges: condition appraisal and remedial treatment, CIRIA, London.
- [4] <http://www.fhwa.dot.gov/bridge/steel/01.cfm>
- [5] Kühn, B., Nussbaumer, A., Helmerich, R. et al. 2004, Recommendation for the fatigue assessment of existing steel structures, ECCS, TC6.
- [6] Bridge Inspection Procedure Manual, 2007, Roads and Traffic Authority of New South Wales, <http://www.directory.nsw.gov.au>
- [7] Yazdani, N., Concrete Bridge Deterioration, SEI-ACI Committee 343 on Concrete Bridge Design.
- [8] <http://www.understanding-cement.com/>
- [9] Virmani, P., Hooks, J. M., Mitigation of corrosion in concrete Bridges, Federal Highway Administration, McLean, VA, <https://www.pwri.go.jp>.
- [10] Madaj, A., Wołowicki, W., 2007, Budowa i utrzymanie mostów.
- [11] <http://theconstructor.org/concrete/sulphate-attack-in-concrete-and-its-prevention/2162/>
- [12] Bridge Inspection, Maintenance, and Repair, 1994, Departments of the Army and the Air Force, ARMY, Report: TM 5-600/ AFJPAM 32-1088.
- [13] Ryall, M. J., Parke, G. A. R., Harding, J. E., 2000, Manual of Bridge Engineering, Thomas Telford.

D 2.1 – Appendix 3

State of the art on damage diagnosis system





D 2.1 – Appendix 3

State of the art on damage diagnosis system

Version 2, 20 February 2017

Colophon

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1 Introduction

Due to a wide variety of unforeseen conditions and circumstances, it will never be possible or practical to design and build a structure that has a zero percent probability of failure. Structural aging, environmental conditions, and reuse are examples of circumstances that could affect the reliability and the life of a structure. There are needs of periodic inspections to detect deterioration resulting from normal operation and environmental attack or inspections following extreme events, such as earthquakes and hurricanes [1].

According to Housner et. al. (1997), structural health monitoring is defined as “the use of in-situ, non-destructive sensing and analysis of structural characteristics, including the structural response, for detecting changes that may indicate damage or degradation”. Nevertheless, the research is mostly focused on data collection and not on evaluation. What is needed is an efficient method to collect data from a structure in service and process the data to evaluate key performance measures, such as serviceability, reliability, and durability. The previous definition should be modified as “the use of in-situ, nondestructive sensing and analysis of structural characteristics, including the structural response, for the purpose of identifying if damage has occurred, determining the location of damage, estimating the severity of damage and evaluating the consequences of damage on the structures” [1].

Structural health monitoring research can be categorized into the following four levels: (I) detecting the existence of damage, (II) finding the location of damage, (III) estimating the extent of damage, and (IV) predicting the remaining fatigue life. Tasks in level (III) require refined structural models and analyses, local physical examination, and/or traditional non-destructive evaluation (NDE) techniques. To perform tasks in level (IV) material constitutive information at local level, material’s aging studies, damage mechanics and high-performance computing are needed [1, 2].

Figure 1, taken from the Research Report 6672-818 of the Department of Civil & Environmental Engineering of the Florida International University, illustrates the basic and main components of structural health monitoring systems:

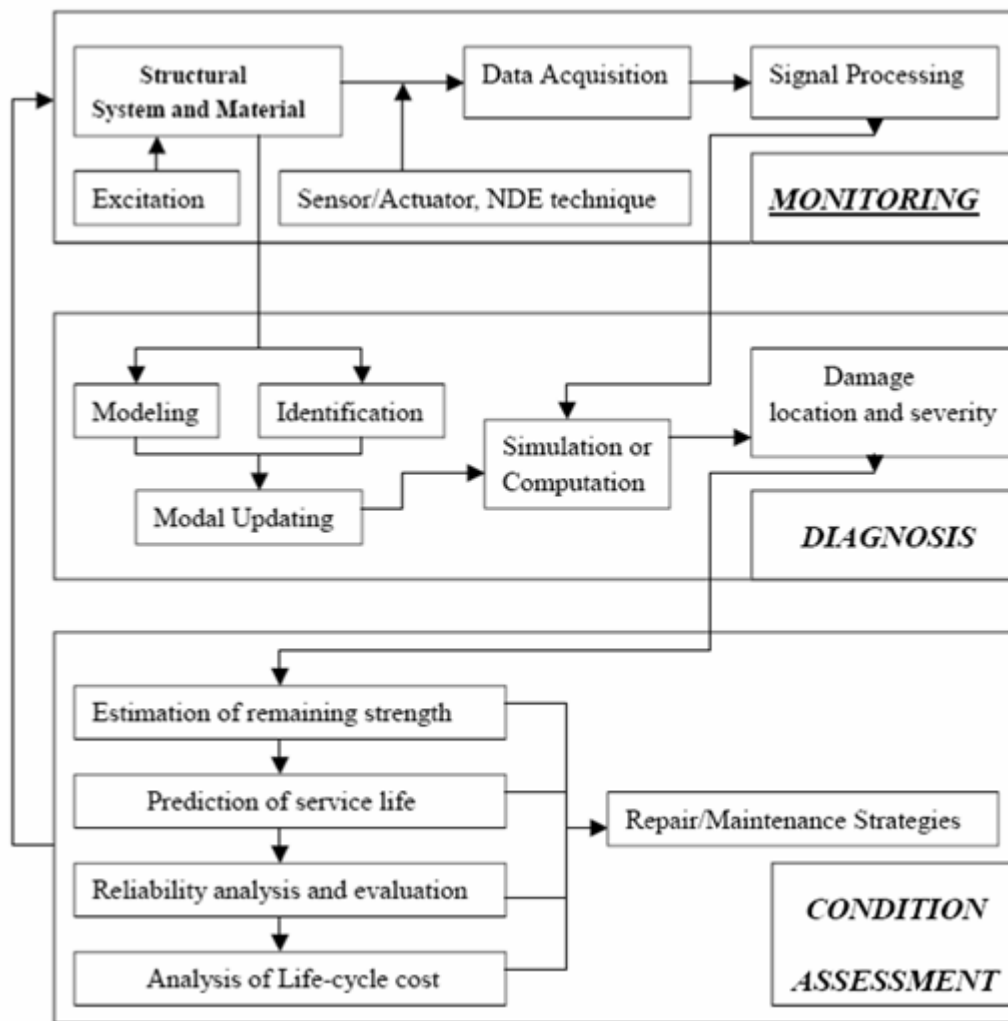


Figure 1 Basic components of structural health monitoring systems

The present document aims to explain the definition of damage in a structure and to summarize and describe the principal damage diagnosis methods, normally used by owners of infrastructure. The main objective of these methods is to identify the existence of damage, its location in the structure, the type of damage found and its severity; in order to diagnose the damage on time and repair it. Damage not treated according to its severity, might lead to loss of human lives.



2 Definition of damage

In the most general terms, damage can be defined as changes introduced into a system that adversely affects its current or future performance. Implicit in this definition, is the concept that damage is not meaningful without a comparison between two different states of a system, one of which is assumed to represent the initial, and often undamaged, state. This review is focused on the study of damage identification in structural and mechanical systems. Therefore, the definition of damage will be limited to changes to the material and/or geometric properties of these systems, including changes to the boundary conditions and system connectivity, which adversely affect the current or future performance of these systems. As an example, a crack that forms in a mechanical part produces a change in geometry that alters the stiffness characteristics of the part. Depending on the size and location of the crack, and the loads applied to the system, the adverse effects of this damage can be either immediate or may take some time before they alter the system's performance. In terms of length scales, all damage begins at the material level and then under appropriate loading scenarios progresses to component and system level damage at various rates. In terms of time scales, damage can accumulate incrementally over long periods of time such as that associated with fatigue or corrosion damage accumulation. Damage can also result from scheduled discrete events such as aircraft landings and from unscheduled discrete events such as an earthquake or enemy fire on a military vehicle [3].

The basic premise of most damage detection methods is that damage will alter the stiffness, mass, or energy dissipation properties of a system, which in turn alter the measured dynamic response of the system. Although the basis for damage detection appears intuitive, its actual application poses many significant technical challenges. The most fundamental challenge is the fact that damage is typically a local phenomenon and may not significantly influence the lower frequency global response of a structure that is normally measured during vibration tests. Stated another way, this fundamental challenge is similar to that found in many engineering fields where there is a need to capture the system response on scales of widely varying length, and such system modeling has proven difficult. Another fundamental challenge is that in many situations damage detection must be performed in an unsupervised learning mode. Here, the term unsupervised learning implies that data from damaged systems are not available. These challenges are supplemented by many practical issues associated with making accurate and repeatable dynamic response measurements at a limited number of locations on complex structures often operating in adverse environments [3].

Environmental and operational variations, such as varying temperature, moisture, and loading conditions affecting the dynamic response of the structures cannot be overlooked either. In fact, these changes can often mask structural changes caused by damage. For instance, Farrar et al. (1994) performed vibration tests on the I-40 Bridge over the Rio Grande in New Mexico, USA, to investigate if modal parameters can be used to identify structural damage within the bridge. Four



different levels of damage were introduced to the bridge by gradually cutting one of the bridge girders as shown in Figure 2. The change of the bridge's fundamental frequency is plotted with respect to the four damage levels as shown in Figure 3. Because the magnitude of the bridge's natural frequency is proportional to its stiffness, the decrease of the frequency is expected as the damage progresses. However, the results in Figure 3 belie the intuitive expectation. In fact, the frequency value increases for the first two damage levels and then eventually decreases for the remaining two damage cases. Later investigation revealed that, besides the artificially introduced damage, the ambient temperature of the bridge played a major role in the variation of the bridge's dynamic characteristics [3].

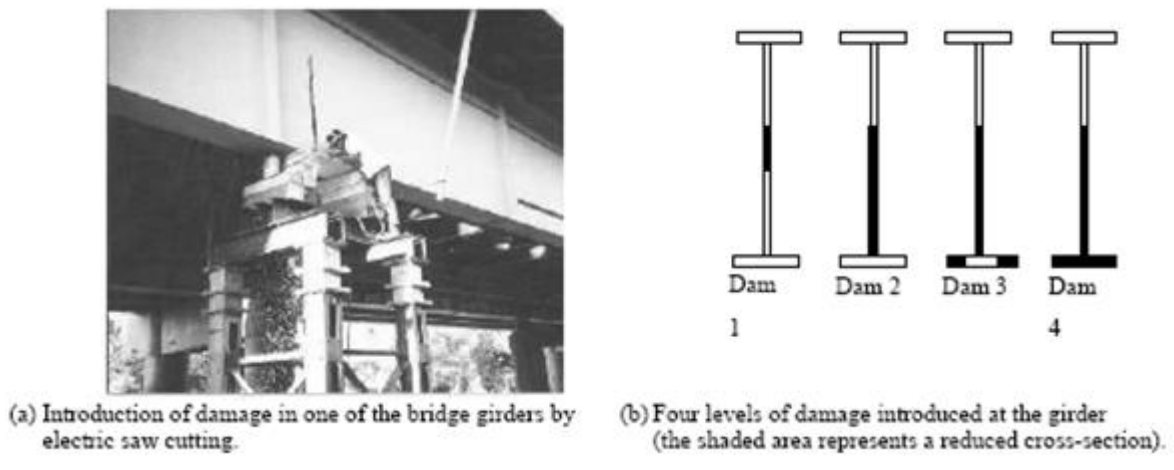


Figure 2 Damage detection study on the I-40 Bridge over the Rio Grande in New Mexico, USA.

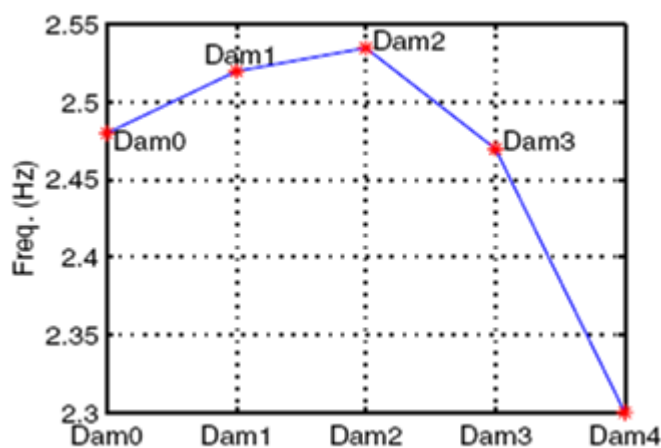


Figure 3 The fundamental frequency change of the I-40 Bridge as a function of the four damage levels shown in Figure 2

In the period 1989-1997, AIC carried out an experimental campaign to determine the causes of collapse of some deck panels of the pier of the “Portosole Sanremo” marina, Liguria, Italy. The



deck panels were made of pre-stressed reinforced concrete with three longitudinal ribs and an upper slab. Exposure to the marine environment with dry/wet cycles had attacked the thin concrete cover and damaged the internal steel wires of some panels.

The experimental dynamic survey showed that all deck panels not affected by pronounced degradation vibrated with a fundamental frequency of about 5 Hz and a second frequency of 9 Hz (Figure 4). This finding was confirmed by a numerical analysis on a FEM model (Figure 5). The analysis also showed that the rupture of a pre-stressing cable in the central or side rib caused a reduction of approximately 16% in the apparent bending stiffness and of about 8% in the fundamental frequency.

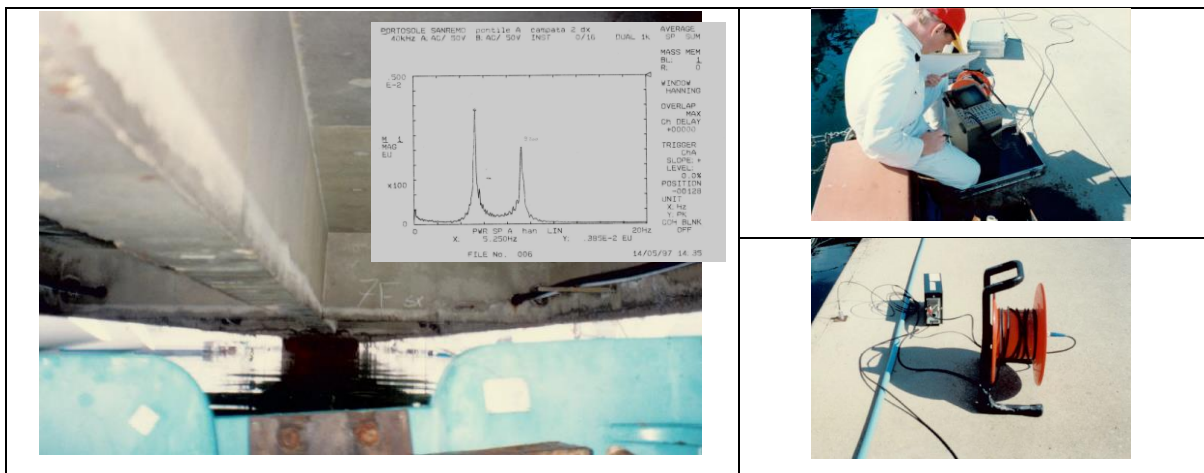


Figure 4 Damage detection study on San Remo Portosole dock in Italy – Experimental dynamic acquisition

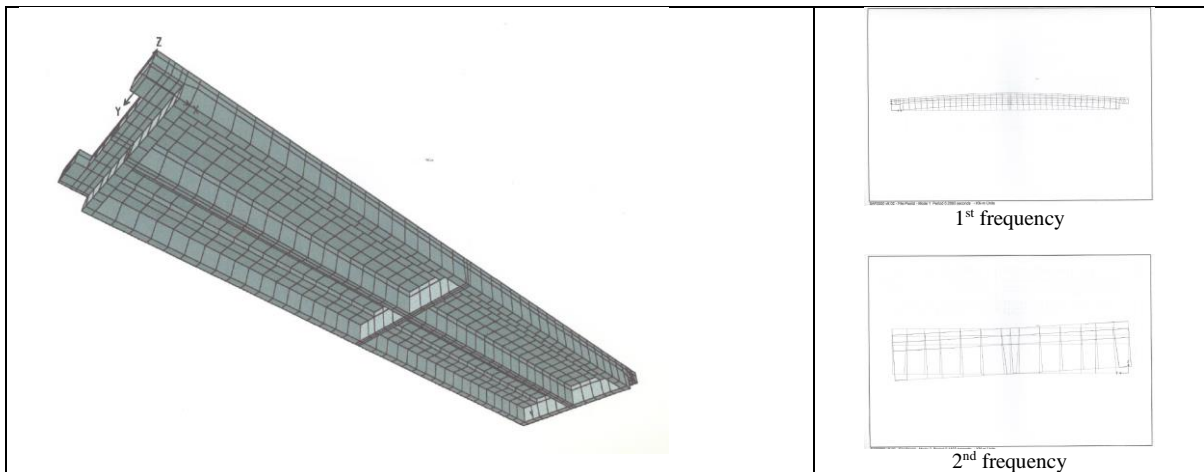


Figure 5 Damage detection study on San Remo Portosole's dock in Italy – FEM models

Experimental dynamic acquisition of the natural frequencies of all the panels allowed the identification of those elements, whose frequency response did not reflect the expected values. In particular, all deck panels having fundamental frequencies less than 4.7 Hz for the first mode or 8.6 Hz for the second mode were classified as “damaged” and thus “unsuitable to sustain the design



loads”. The predictions made turned out to be accurate, as some of the deck panels classified as “unsuitable” actually collapsed after some time.

2.1 Damage and degradation mechanisms

Taking into account the nature of degradation processes, the following main groups of degradation mechanisms are distinguished [4]:

- Chemical mechanisms: causing degradation of bridge structures as a result of chemical reactions.
- Physical mechanisms: diminishing condition of bridge structures by influence of physical phenomena;
- Biological mechanisms: reducing condition of bridge structures by influence of biological phenomena.

The proposed taxonomy of basic degradation mechanisms identified in the case of concrete railway bridges is presented in Figure 6.

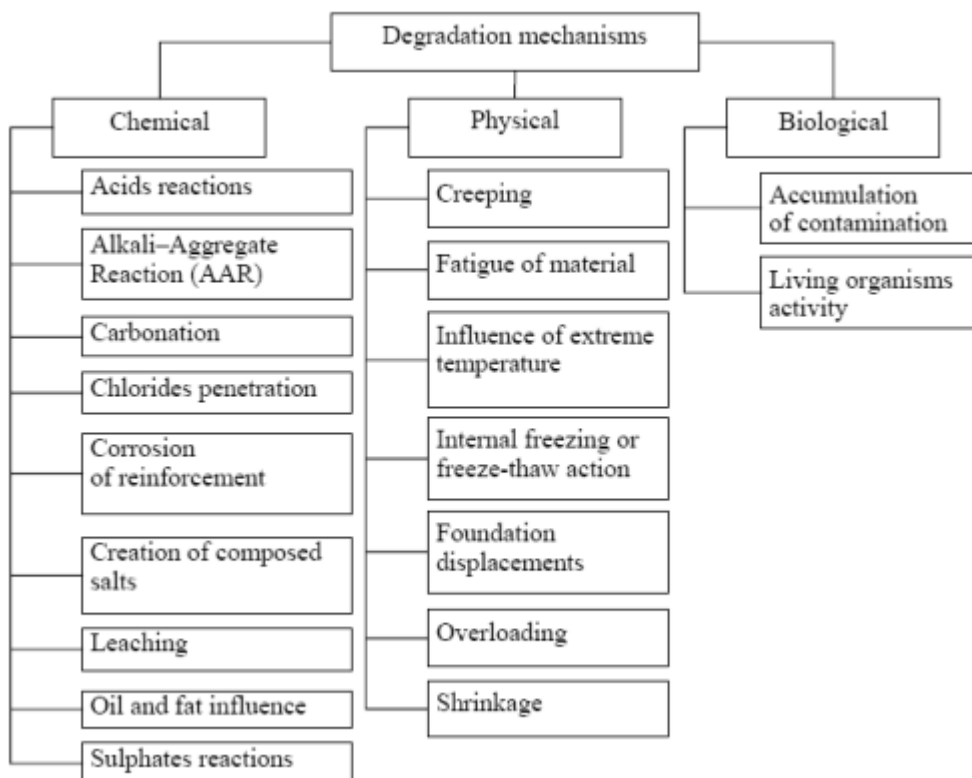


Figure 6 Degradation mechanisms of concrete railway bridges [4]

The most frequent chemical degradation mechanisms can be defined as following [4]:



- Acids reactions: corrosion of concrete caused by the reaction of the concrete components with the aggressive acid environment at the external layer of concrete. This reaction is caused by: presence of carbon dioxide, sulfates, ammonium or magnesium and also strong mineral acids like: sulphuric, hydrochloric, nitric even diluted. The factors which increase activity of this mechanism are: hydrated cement and poor maintenance (organic materials can release sulphuric acid).
- Alkali – Aggregate Reaction (AAR): the mechanism caused by presence of aggregates and alkali, which leads to an expansive reaction and destruction of the concrete.
- Carbonation: mechanism where carbon dioxide, from the atmosphere, enters to concrete and reacts with the hydroxides to form carbonates and water. Due to the consumption of these hydroxides the pH value is reduced below 9.0. The straight cause of carbonation is the presence of CO₂ in the surrounding air and porosity of concrete. The factors which increase ability of concrete to carbonate are: lower content of CaO, higher diffusion constant of CO₂, higher w/c value, cracks, lower strength of concrete, presence of mineral additives and also the lack of curing of concrete in moist environment.
- Chlorides penetration: losing the passivation causes by presence of chloride ions, which locally breaks down the passive film. Chlorides mechanism is caused by: marine sea environment – permanent penetration of chlorides and de-icing salts. The factors which increase activity of this mechanism are: high value of w/c, carbonization and moisture, low concentration level of OH(-), mineral additives like: silica dust, fly-ash, blast furnace slag, lower content of C3A, the curing of concrete in moist environment even of small salt content, the salting on the construction surfaces subjected to the activity of the atmospheric factors.
- Corrosion of reinforcement: the oxidation of steel initiated by the losing of rebars passivation at the high electrolytic conductivity of concrete (high humidity) and the permanent supply of oxygen from outside. The expansive character of producing oxides induces cracks and loss of the concrete cover. The factors which increase activity of this mechanism are: carbonization of concrete, chlorides and cracks.
- Creation of composed salts: mechanism caused by the reaction of aggressive substances from the surrounding environment with the concrete components. This mechanism causes creation of the salts in concrete pores. When the salts crystallize, they enlarge in volume. This leads to increase in tensile stress and cause destruction of the pore walls and strength reduction. The partial humidity of concrete and the possibility of the temporary steaming of water, increase the activity of this mechanism.
- Leaching: destruction of concrete caused by soft water. The components of concrete are dissolved and leached. The mechanism is caused by water, especially clear, distilled and



condensation and also by the mountain streams and precipitations. Factors which increase activity of this mechanism are: cracks, higher value of w/c index, higher content of $\text{Ca}(\text{OH})_2$ and higher permeability of concrete.

- Oil and fat influence: reaction of oils and/or fats with the calcium hydroxide. During this reaction calcium soaps are created, which are the greasy substances and do not have binding features.
- Sulphates reactions: corrosion of concrete caused by the reaction of concrete components with aggressive sulphate environment like water, contaminated groundwater, soils, seawater, decaying organic substance or industrial effluent. This mechanism can be caused also by or aggregate that contains sulphates.

In physical mechanisms category, the following main processes cause degradation of concrete bridge [4]:

- Creep: inelastic strains caused by long-time load.
- Fatigue of material: mechanisms of sequential degradation of material, by crack initiation followed by increasing to critical size, from which unstable escalation is started. This phenomenon is caused by recurrent loads.
- Foundation displacements: mechanism causing changes in the global geometry of the structure. This phenomenon leads to adverse redistribution of internal forces and consequently – stresses, which can exceed designed values of the load capacity and cause serious damage. This mechanism can be caused by floods, mine activity, earthquakes or wrong parameters of the ground taken into consideration in design.
- Influence of high temperature: phenomenon caused for example by a rail or truck mounted tanker getting ignited accidentally in the vicinity of a bridge.
- Internal freezing or freeze-thaw action: mechanism caused by the expansion of pore water due to freezing. This mechanism causes deep cracks. Factors which increase the effects of this mechanism are: the w/c ratio > 0.6 ; decreased air content; continuous storing of concrete in permanent contact with water without drying time; fine porous aggregate; alkalis content; mineral additives like cinder, fly-ash without aerating substances; cracks; structures sucking ground water; railway bridge troughs filled with moist ballast, etc.
- Overloading: exceeding of the acceptable designed values of the load. Overloading can be caused by: changing of the load class; floods, earthquakes, collisions or any other accident.
- Salt-frost scaling: mechanism caused by the expansion of pore water due to freezing in the presence of salt water; factors which increase the effects of this mechanism are: the w/c ratio > 0.4 ; lower air content in fresh concrete before the cementation; presence of silica



dust; continuous storing of concrete in permanent contact with water without drying time; fine porous aggregate; leaching; acid attack.

- Scour of foundation: mechanism caused by the moving liquid (for instance river) through the support zone and causing losses of support material as well as the soil.
- Shrinkage: This mechanism is caused by the internal constraint of element deformation by reinforcement resistance. It is caused also by the external constraint of element movement by supports and can be caused by badly performed curing of concrete exposed to the sunbeams.

Biological degradation mechanisms form the smallest but important group of processes diminishing condition of concrete bridges. The following main processes can be listed [4]:

- Accumulation of contamination: mechanism of organic and non-organic contaminants increase caused by environmental and human activities:
- Living organisms' activity: mechanism causing damage as a result of living organisms' activity like bacteria, plants and animals.

All described degradation mechanisms can be also classified taking into account duration of the degradation processes. The following groups can be distinguished:

- Incidental mechanisms: when the degradation process is very short (duration even below a second), e.g. overloading by collision or by earthquake.
- Short-time mechanisms: acting during hours or days, e.g. influence of extreme fire temperature, foundation displacement because of scour during flood.
- Long-time mechanisms: majority of considered chemical, physical and biological processes.

Relationships between degradation mechanisms and the basic types of damage, based on analysis of many practical cases, are presented in Table 1. Identified damage is usually caused by more than one degradation mechanism and correct understanding of damage processes needs very precise analysis [4].



Table 1 Degradation mechanisms and basic types of damages

Degradation mechanisms Damages	Chemical								Physical							Biological				
	Acids	Alkali – Aggregate Reaction (AAR)	Carbonation	Chlorides penetration	Corrosion of reinforcement	Creation of composed salts	Leaching	Oil and fat influence	Sulphates reactions	Creeping	Fatigue of material	Influence of high temperature	Internal freezing	Foundation displacements	Overloading	Salt-frost scaling	Scour of foundation	Shrinkage	Accumulation of contamination	Living organisms activity
Destruction	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓
Discontinuity		✓	✓	✓	✓	✓		✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓		✓
Losses	✓	✓	✓	✓	✓			✓		✓	✓	✓	✓	✓	✓	✓	✓		✓	✓
Deformations									✓		✓			✓	✓		✓	✓		
Displacements													✓	✓		✓			✓	
Damages of protection	✓	✓	✓	✓		✓	✓	✓	✓	✓	✓	✓		✓	✓	✓	✓	✓	✓	✓
Contaminations		✓			✓		✓				✓	✓				✓			✓	✓



3 Damage diagnosis methods

Typically, the assessment process of an existing structure consists of the following steps [5]:

- Preliminary on-site inspection (to ascertain location, condition, loadings, environmental influences, necessity for further testing)
- Recovery and review of all relevant documentation, including loading history, maintenance and repair and alterations
- Specific on-site testing and measurements (e.g. proof loading)
- Analysis of collected data to refine the probabilistic models for structural resistance
- Accurate (re-)analysis of the structure with updated loading and resistance parameters
- Structural reliability analysis
- Decision analysis

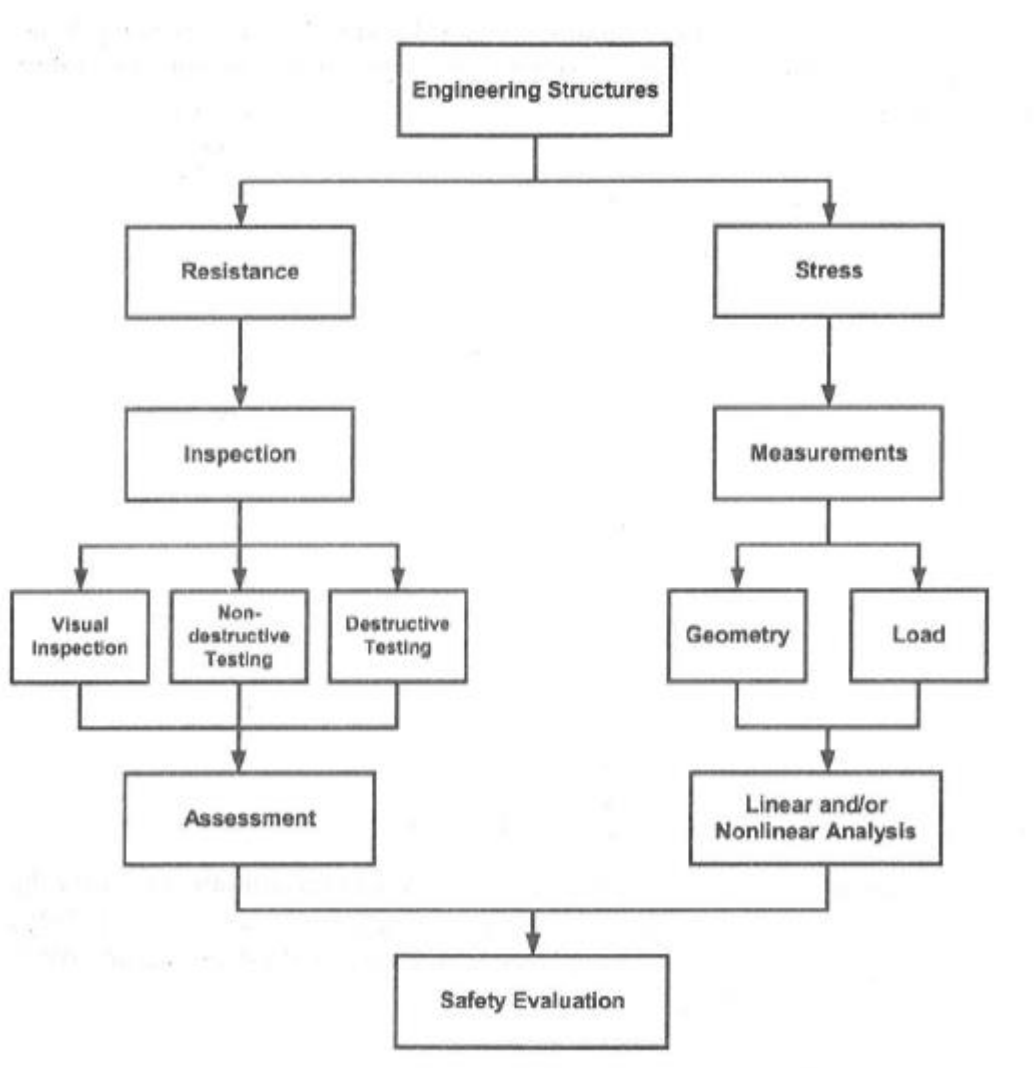


Figure 7 Structural assessment



Typical test methods used for concrete structures are shown in Table 2 [5]. Some of them, especially the Non Destructive Techniques, are described further on.

Table 2 Test methods for concrete structures

Property under investigation	Test
Material properties	<ul style="list-style-type: none"> Cores Pull-out Pull-off Break-off Internal fracture Penetration resistance Rebound hammer Maturity
Concrete quality, durability and deterioration, reinforcement	<ul style="list-style-type: none"> Surface hardness Ultrasonic pulse velocity Radiography Radiometry Neutron absorption Relative humidity Permeability Absorption Petrography Sulphate content Expansion Air content Cement type and content Abrasion resistant
Corrosion	<ul style="list-style-type: none"> Half-cell potential Resistivity Linear polarization resistance A.C. Impedance Cover depth Carbonation depth Chloride concentration
Integrity and performance	<ul style="list-style-type: none"> Tapping Pulse-echo Dynamic response



	Acoustic emission Thermoluminescence Thermography Radar Strain or crack measurement Load test
Element dimensions	Reinforcement location Electromagnetic cover meters Ultrasonics
Deformation and displacements	Extensometer Dial gages Potentiometer LVDT Fiber optic sensors Geodetic measurements GPS Laser Strain measurement Heave or settlement of supports Deflection Liquid leveling systems Slope measurement
Loads	WIM systems Load cells
Dynamic response	Velocity and acceleration measurement Dynamic load tests

3.1 Visual inspection

The visual inspection is the first step necessary for the condition assessment of structures. By means of visual inspection an overall impression should be obtained of all symptoms of deterioration including the identification of actual and potential sources of trouble. All the activities leading to the final choice of a rehabilitation strategy for a damaged bridge are initiated at this stage [5]. In order to ensure an optimum collection of information an inspection concept is needed, including [5]:

- Description of the structure



- Historical information (including previous inspection reports)
- Access equipment needed (tower wagon or scaffold) – lane closure, necessary downtimes
- Possible removal of everything that prevents good visual access
- Inspection equipment
- Competences & responsibilities

Inspections which may significantly interfere with normal traffic movement and might affect the safety of the inspectors must be coordinated with district personnel in order to perform appropriate traffic control measures to be undertaken. Inspections of the underside of bridges that cannot be reached by conventional ladders may be performed by the use of vehicles with under-bridge platforms [5].

During the visual inspection special attention needs to be paid to various factors, including:

- Verification of information gathered during the planning of the assessment
- Old coatings, impregnations or protections
- The appearance of the original concrete surface
- Differences of the color of the concrete surface
- The presence of cracks, their appearance and pattern
- Superficial deterioration of the concrete skin
- Deterioration of the concrete itself
- Exposed rebar
- Deformations of the structure
- Presence of humidity or water, leakages, etc.
- Fouling (algae, moss, tresses)

The findings must be described in detail as they form the basis for any consequent measures. In addition, it may be useful to state the name of the inspector, as well as the names of all that may have been attendance [5].

3.1.1 Routine inspection

The routine inspection is a visual inspection of all visible parts of the structure. The inspection should be carried out by a highly experienced bridge engineer. The purpose is to maintain an overview of the general condition of the whole infrastructure stock, and to reveal significant damage in due time, taking safety and economic aspects into consideration.

One set of information generated during a routine inspection is a series of “condition ratings” assigned to various structural components. These condition ratings give an overall measure of the condition of a structure by considering the severity of deterioration in the structure and the extent



to which it is distributed throughout each component. The ratings assigned to each element are based on a standard set of definitions associated with numerical ratings [5].

The equipment needed for routine bridge inspections usually includes the following [5]:

- Cleaning tools including wire brushes, screwdrivers, brushes, scrapers, etc.
- Inspection tools including pocketknife, ice pick, hand brace, bit, and increment borer for boring timber elements, chipping hammer, etc.
- Visual aid tools including binoculars, flashlight, magnifying glass, dye penetrant, mirror, etc.
- Basic measuring equipment including thermometer, center punch, simple surveying equipment, etc.
- Recording materials such as appropriate forms, field books, cameras, etc.
- Safety equipment including rigging, harnesses, scaffolds, ladders, bosun chairs, first-aid kit, etc.
- Miscellaneous equipment should include C-clamps, penetrating oil, insect repellent, wasp and hornet killer, stakes, flagging, markers, etc.

Common concrete member defects include cracking, scaling, delamination, spalling, efflorescence, wear or abrasion, collision damage, scour, and overload. The inspection of concrete should include both visual and physical examination.

Common steel and iron member defects include corrosion, crack, collision damage, and overstress. Cracks usually initiate at the connection detail, at the termination end of a weld, or at corroded location of a member and then propagate across the section until the member fractures. Since all the cracks may lead to failure, bridge inspectors need to look at each and every one of these potential crack locations carefully [5].

3.1.2 In-depth inspection

In depth inspections are usually performed as a follow-up inspection to a Routine Inspection to better identify any deficiencies found. A testing program for reinforced concrete structures that supplements visual observations may include obtaining and testing cores and samples for compressive strength, chloride ion content, depth of carbonation, pH value, and petrographic examinations. Load testing may also sometimes be performed as part of an In-Depth Inspection [5].

3.1.2.1 Concrete strength

The compressive strength of concrete in existing structures can be determined by testing drill cores or non-destructively by performing sclerometer tests. Testing drill cores is the most direct and reliable way of concrete strength testing. However, this method is labor-intensive and partially damaging the structure. After removing the drill core, the remaining hole needs to be closed carefully in order to avoid further damage. It is recommended to use epoxy or cement mortar.



Alternatively pull-off tests can be performed to estimate the concrete strength [5]. The pull-off test has been developed to measure the in-situ tensile strength of concrete by applying a direct tensile force, see Figure 8. The test can be executed on site. The choice of test locations can be based on results of the visual inspections or by the results of the concrete surface hardness test. The results of the hammer sounding may indicate areas in which spalling occurs. In these areas pull-off tests have no use at all [5].



Figure 8 Pull-off test

3.1.2.2 Steel strength

In the case that specimens from the structure are available, tests to determine the static tensile strength, relaxation, fatigue and corrosion can be performed in order to obtain the main material properties of the steel. In the case of post-tensioning tendons which are structural elements essential for the safety, serviceability and durability of a pre-stressed concrete structure; non-destructive techniques which allow for a reliable assessment of the properties of the post-tensioning steel without damaging the structure are very desirable [5].

3.1.2.3 Carbonation depth

For evaluating the possibility of corrosive damage to the surface it is necessary to measure the depth at which the carbonation has decreased the pH value below the critical level. There are two ways for determining the carbonation depth [5]:

- A new fracture surface (perpendicular to the external concrete surface) is sprinkled by a liquid indicator, preferably Phenolphthalein. In the basic region the indicator changes its color into a pink shade. The line between the colorless and the pink surface marks the carbonation depth. The fracture surface can be obtained by splitting a drill core. It is not advisable to use the surface of the drill core because the $\text{Ca}(\text{OH})_2$ is being smeared over the core surface during the wet drilling process.
- Using a standard drill bit some pulverized material is removed from the structure and preserved in distinct portions according to the drilling depth. Then the material is mixed



with water and the mixtures pH-values are determined. The drill bit method is recommended for determining the pH-value of the grouting mortar in tendon ducts too.

3.1.2.4 Chloride determination

For determining the chloride content a pulverized sample is exposed into an acid solution and then a quantitative analysis is carried out. The acid solution is necessary for dissolving chlorides out of water resistant salts. By doing this a conservative value of the chloride content is determined which even after carbonation of the concrete will not be exceeded. The allowable value amount is from 0.2% to 0.4% of the cement mass [5].

3.2 Non-destructive evaluation (NDE)

Structures are checked for any sudden damage or deterioration such as signs of settlements or displacements, damage on slabs, girders, columns or piers due to impact from traffic, erosion of slopes, etc. For registration of any kind of failure or damage observed, specially prepared forms are filled in, photos taken and the material handed over to the responsible engineer for further action [5].

Assessing the structural condition without removing individual structural components is known as non-destructive evaluation (NDE) or nondestructive inspection. NDE techniques include those involving acoustics, dye penetrating, eddy current, emission spectroscopy, fiber-optic sensors, fiber-scope, hardness testing, isotope, leak testing, optics, magnetic particles, magnetic inspection, etc. Most of these techniques have been used successfully to detect location of certain elements, cracks or weld defects, corrosion/erosion, and so on [1].

Most structural health monitoring methods under current investigation focus on using dynamic responses to detect and locate damage because they are global methods that can provide rapid inspection of large structural systems. These dynamics-based methods can be divided into four groups: (1) spatial-domain methods, (2) modal-domain methods, (3) time-domain methods, and (4) frequency domain methods. Spatial-domain methods use changes of mass, damping, and stiffness matrices to detect and locate damage. Modal-domain methods use changes of natural frequencies, modal damping ratios, and mode shapes to detect damage. In the frequency domain method, modal quantities such as natural frequencies, damping ratio, and modal shapes are identified. The reverse dynamic system of spectral analysis and the generalized frequency response function estimated from the nonlinear auto-regressive moving average (NARMA) model are applied in nonlinear system identification. In time-domain method, system parameters are determined from the observational data sampled in time. It is necessary to identify the time variation of system dynamics characteristics from time domain approach if the properties of structural system change with time under the external loading condition. Moreover, one can use model-independent methods



or model-referenced methods to perform damage detection using dynamic responses presented in any of the four domains. Literature shows that model-independent methods can detect the existence of damage without much computational efforts, but they are not accurate in locating damage. On the other hand, model-referenced methods are generally more accurate in locating damage and require fewer sensors than model-independent techniques, but they require appropriate structural models and significant computational efforts. Although time-domain methods use original time-domain data measured using conventional vibration measurement equipment, they require certain structural information and massive computation and are case sensitive. Furthermore, frequency- and modal-domain methods use transformed data, which contain errors and noise due to transformation. Moreover, the modeling and updating of mass and stiffness matrices in spatial-domain methods are problematic and difficult to be accurate. There are strong development trends that two or three methods combined together to detect and assess structural damage. The combination could remove the weakness of each method and check each other. It suits the complexity of damage detection [1].

Table 3 [4] presents the application range of selected NDT methods applied for identification of basic damage types common in concrete bridges.

Table 3 Application of NDT methods for identification of concrete bridge damages

NDT methods	Damages						
	Destruction	Discontinuity	Losses	Deformations	Displacements	Damages of protection	Contaminations
1	2	3	4	5	6	7	8
Acoustic and optic methods							
Acoustic emission	<input type="checkbox"/>	<input type="checkbox"/>					
Endoscopy		<input type="checkbox"/>	<input type="checkbox"/>			<input type="checkbox"/>	
Fiber optic sensors		<input type="checkbox"/>		<input type="checkbox"/>			
Impact-echo	<input type="checkbox"/>	<input type="checkbox"/>				<input type="checkbox"/>	
Low strain Pile Integrity Testing	<input type="checkbox"/>	<input type="checkbox"/>					
SASW	<input type="checkbox"/>						
Ultrasonic tomography/test	<input type="checkbox"/>	<input type="checkbox"/>				<input type="checkbox"/>	
Electrical/electro-magnetic methods							
Potential Mapping	<input type="checkbox"/>		<input type="checkbox"/>			<input type="checkbox"/>	
electromagnetic test	<input type="checkbox"/>	<input type="checkbox"/>				<input type="checkbox"/>	
Galvapulse			<input type="checkbox"/>				



Ground penetrating radar		<input type="checkbox"/>					
Impulse Radar	<input type="checkbox"/>	<input type="checkbox"/>					
Geodetic measurements							
Global Positioning System (GPS)				<input type="checkbox"/>	<input type="checkbox"/>		
Laser displacement measurements				<input type="checkbox"/>	<input type="checkbox"/>		
Triangulation				<input type="checkbox"/>	<input type="checkbox"/>		
Load tests							
Static load tests	<input type="checkbox"/>						
Dynamic load tests	<input type="checkbox"/>	<input type="checkbox"/>					
Vibration tests	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>				
Radiographic methods							
Radiography with x-ray/cobalt		<input type="checkbox"/>					
Spectral-chemical and potential methods							
Laser induced breakdown spectroscopy	<input type="checkbox"/>						
Test of pollution	<input type="checkbox"/>					<input type="checkbox"/>	
Thermal heat transfer							
Pulse-phase thermography		<input type="checkbox"/>					
Transient thermography	<input type="checkbox"/>	<input type="checkbox"/>					
Visual and simple methods							
Covermeter						<input type="checkbox"/>	
Direct measurement		<input type="checkbox"/>	<input type="checkbox"/>			<input type="checkbox"/>	<input type="checkbox"/>
Displacement gauges					<input type="checkbox"/>		
Hydrostatic levelling system (HLS)				<input type="checkbox"/>			
Mechanical sensors				<input type="checkbox"/>			
Roughness depth test	<input type="checkbox"/>					<input type="checkbox"/>	
Sclerometer test	<input type="checkbox"/>						
Strain measurement	<input type="checkbox"/>			<input type="checkbox"/>			
Tapping	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>				
Torrent permeability	<input type="checkbox"/>						
Visual inspection	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>	<input type="checkbox"/>
Void volume measurement		<input type="checkbox"/>	<input type="checkbox"/>				

The most common NDE techniques used for concrete evaluation, reinforcement detection and steel evaluation are described next.

3.2.1 Concrete evaluation



Concrete structures could deteriorate due to heavy traffic loads, fatigue, chemical reactions, unpredictable disasters, and poor workmanship [6]. In this regard, effective inspection techniques become very important prior to repair works. The principal NDE techniques for concrete evaluation are described next.

3.2.1.1 Radiographic testing

It is used for the detection of broken wires in cable-stayed bridges, imaging of post-tensioning strands in concrete beams, and the detection of voids in the grouted post-tensioning ducts.

The system consists of passing x-rays or gamma rays through the member being tested and creating an image on a photosensitive film. If there is a crack in the member or a void in the weld, less radiation is absorbed by the steel and more radiation passes through that area to the film. Thus the defects are shown on the film as dark lines or shaded areas, compared to the surrounding areas of sound material. This method has an advantage of providing a permanent record for every test carried out. However, it requires specialized knowledge in selecting the angles of exposure and also interpreting the results recorded on the films. It also requires access from both sides of the test area, with the radiation source placed on one side and the film placed on the other side [5].

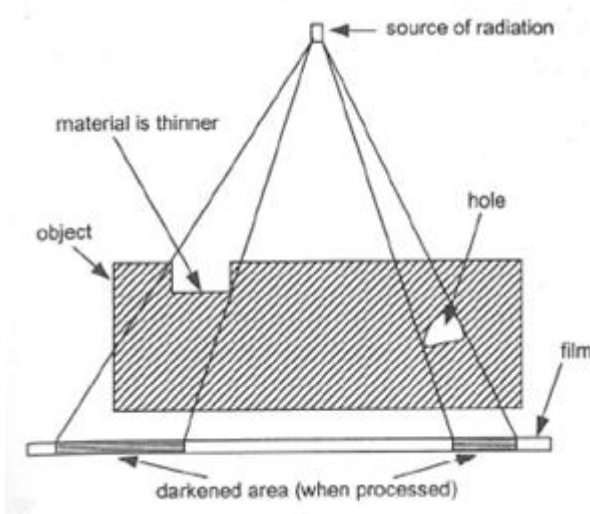


Figure 9 Radiographic testing

3.2.1.2 Ultrasonic pulse echo

This method is based on measuring the time difference between sending of an impulse and receiving the echo (see Figure 10). In addition, the intensity of the echo is measured. An electro-mechanical transducer is used to generate a short pulse of ultrasonic stress waves that propagates into the object being inspected. Reflection of the stress pulse occurs at boundaries separating materials with different densities and elastic properties. The reflected pulse travels back to the transducer that also acts as a receiver.

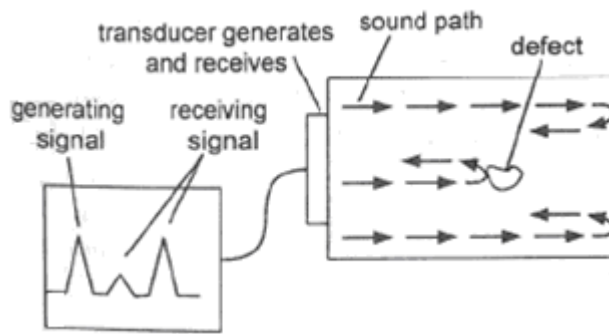


Figure 10 Ultrasonic testing

The receiver signal is displayed on an oscilloscope, and the round trip travel time of the pulse is measured electronically. The results are displayed in a time-position-plot (see Figure 11). If the ultrasonic velocity is known, the time can be related to the location of the flaw causing the echo.

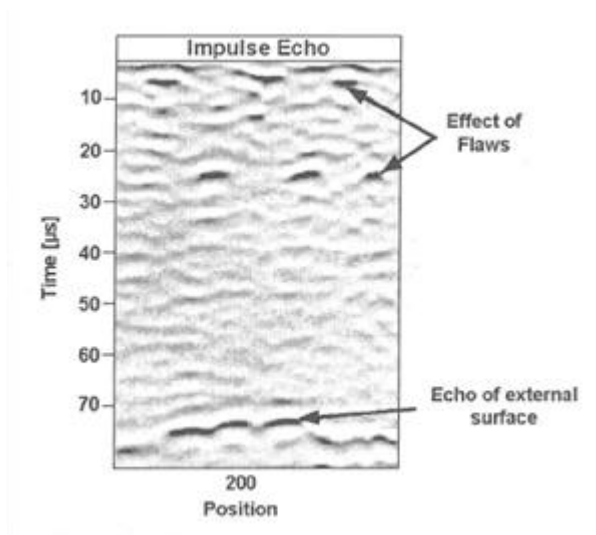


Figure 11 Position-time plot

In nondestructive testing of metals, the ultrasonic pulse-echo (UP-E) technique has proven to be a reliable method for locating cracks and other internal defects. The presence of paste-aggregate interfaces, air voids, and reinforcing steel results in a multitude of echoes that obscure those from real defects. However, for investigating the homogeneity in concrete walls or slabs with constant thickness the measurement of the ultrasonic velocity has proven to be an efficient method [5].

3.2.1.3 Ultrasonic pulse velocity method

This method measures the time of travel of an ultrasonic pulse passing through the concrete. The fundamental design features consist of a pulse generator and a pulse receiver. Pulses are generated by shock-exciting piezo-electric crystals, with similar crystals used in the receiver. The time taken for the pulse to pass through the concrete is measured by electronic measuring circuits. Pulse



velocity tests can be carried out on both laboratory-sized specimens and completed concrete structures, but some factors affect measurement:

- There must be smooth contact with the surface under test; a coupling medium such as a thin film of oil is mandatory.
- It must be recognized that there is an increase in pulse velocity at below-freezing temperature owing to freezing of water; from 5 to 30°C pulse velocities are not temperature dependent.
- The presence of reinforcing steel in concrete has an appreciable effect on pulse velocity. It is therefore desirable and often mandatory to choose pulse paths that avoid the influence of reinforcing steel or to make corrections if steel is in the pulse path.

Ultrasonic pulse velocity tests have a great potential for concrete control, particularly for establishing uniformity and detecting cracks or defects. Its use for predicting stress is much more limited, owing to the large number of variables affecting the relation between strength and pulse velocity.

Fairly good correlation can be obtained between concrete compressive strength and pulse velocity (see Figure 12). These correlations enable the strength of structural concrete to be predicted within $\pm 20\%$, provided the types of aggregate and mix proportions are constant. However, this method should be applied in combination with a destructive method only since it does not provide the required accuracy without calibration. The concrete member to be tested has to be accessible from two opposite sides. For high strength concrete the sensitivity of the method is comparatively small. The applicability of the ultrasonic strength evaluation is limited therefore to special cases only [5].

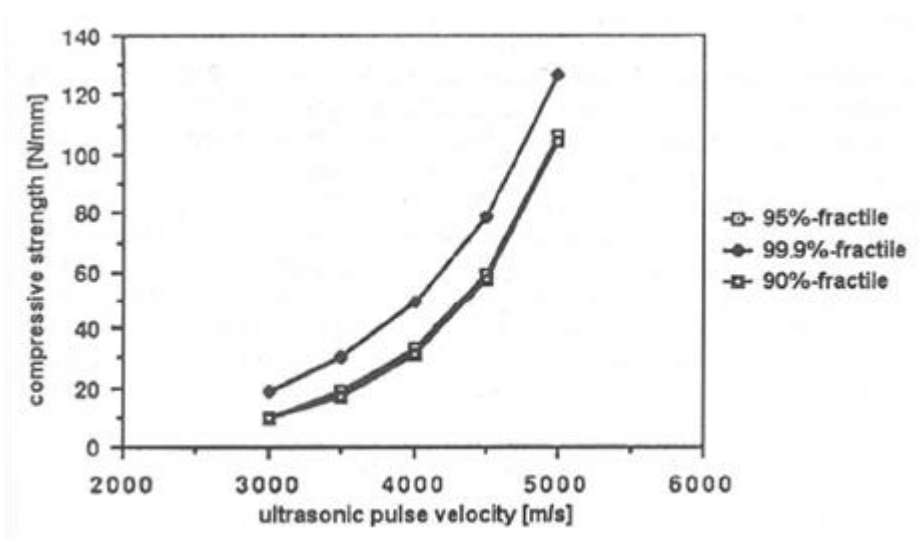


Figure 12 Relation between compressive strength and pulse velocity (Bundesministerium für wirtschaftliche Angelegenheiten, 1987)



3.2.1.4 Impact Echo method

This method is used to find imperfections in concrete. It is based on using a short-duration mechanical impact to generate low frequency stress waves (2 to 20 kHz, typically) that propagate into the structure and are reflected by flaws and external surfaces of the structure. The impact can be produced by tapping a steel ball against the concrete surface or by hitting the surface by using a hammer, see Figure 13.

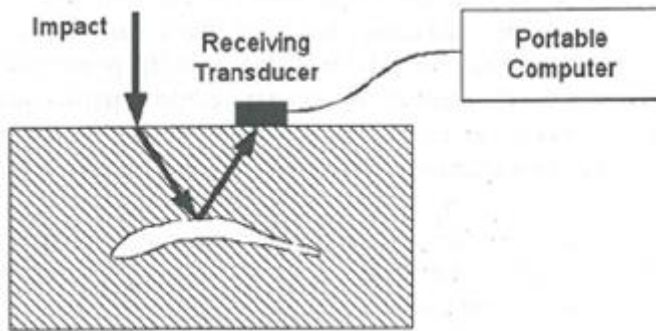


Figure 13 Schematics of the Impact Echo method

As the low frequency stress waves propagate through the structure, they are reflected by air interfaces within the structure and the external surfaces of the structure. Possible air interfaces are: delaminations, voids, and cracks. Multiple reflections of the stress waves, between the impact surface, flaws, and/or other external surfaces, give rise to modes of vibration, which can be identified by frequency and used to determine the geometry of a structure or the location of flaws. A receiver, located on the surface near the location of impact, monitors the surface displacements caused by the arrival of the reflected waves. The record of displacement versus time is transformed into the frequency domain for ease of signal analysis. The presence and nature of any internal flaws or external interfaces can be determined from analysis of time-domain waveform and frequency spectrum [5, 7].

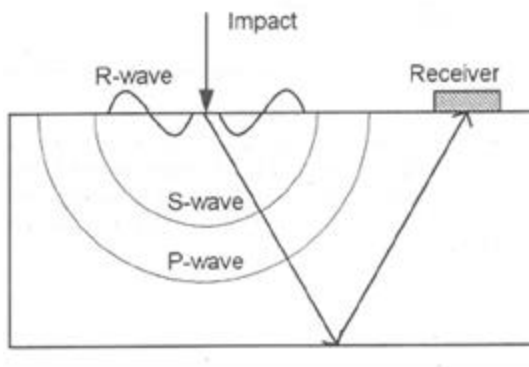


Figure 14 Impact Echo testing

Typical applications of impact-echo testing include [7]:

- Slab thickness measurements



- Detecting delaminations, cracks and voids
- Evaluating unconsolidated concrete
- Locating voids in grouted tendon ducts
- Locating subgrade voids beneath foundation slabs
- Evaluating mine shafts and tunnel liners
- Finding voids in grouted masonry
- Evaluating distributed damage in concrete

3.2.1.5 Infrared thermography (IT)

This is a non-contact optical method, which utilizes differences in heat transfer through a structure to reveal the locations of hidden defects. IT is used to [7]:

- Detect voids in grouted masonry
- Detect delaminations in concrete bridge decks and slabs
- Detect moisture variations in roof and wall systems

However, IT is limited by environmental conditions and has difficulty evaluating decks with asphalt overlays. The dual-band infrared Thermography using two different infrared wavelengths simultaneously overcomes some of the operational problems (primarily surface emissivity variations) encountered with standard infrared Thermography [5].

One of the methods selected for the bridge inspection is an active or transient Thermography. This method is dissimilar to the conventional thermographic methods in the utilization of time-dependent heating (or cooling) of the target. Depending on the type of defect and thermal characteristics of a target, an external heating or cooling is applied in the form of short energy pulses. The created thermal perturbation is then followed by a differential time-resolved infrared image analysis [5, 7].

3.2.1.6 Ground-penetrating radar (GPR) systems

GPR is a pulse-echo method for measuring pavement layer thickness and other properties. It works like ultrasound, but uses radio waves rather than sound waves to penetrate the pavement, see Figure 15 and Figure 16. GPR systems are normally used to locate structural components, like reinforcing bars embedded in concrete, to avoid weakening the structure while collecting core samples for detailed inspection. Advanced GPR, integrated with imaging technologies for use as nondestructive evaluation tool, enables to quickly locate and characterize construction flaws and wear- or age-induced damage in these structures without the need for destructive techniques like coring. The bridge deck and its wearing surface are the most vulnerable parts of a bridge to damage from routine service, and they are particularly well suited for inspection using a vehicle-mounted inspection system. An advanced GPR system can reliably detect, quantify, and image



delaminations in bridge decks. Such a system is designed to operate at normal highway speeds, eliminating the need for lane closure [5].

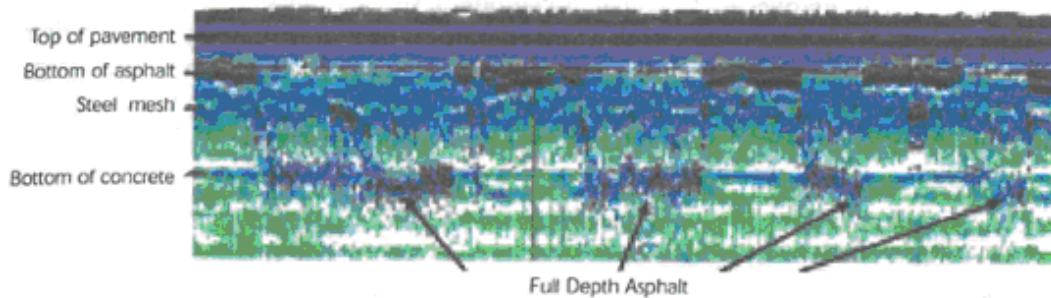


Figure 15 GPR record of asphalt-overlaid concrete, showing evidence of full-depth patching in concrete (www.enviroacoustics.gr)

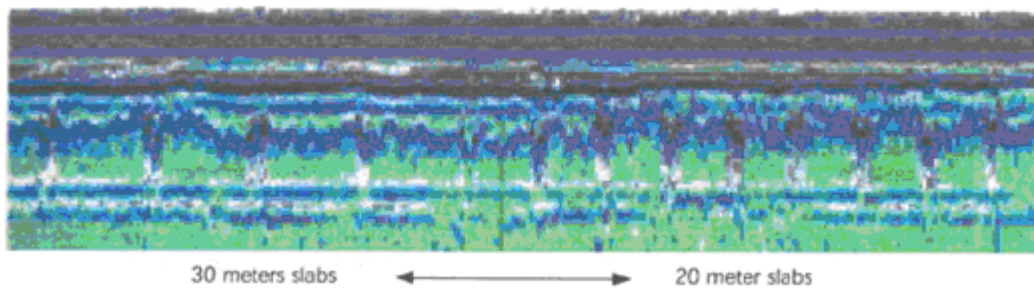


Figure 16 GPR record showing transition in slab length from 30 to 20 meters (www.fhwa.dot.gov)

The basic configuration of a mobile GPR equipment includes at least one radar antenna mounted on a GPR vehicle with data acquisition unit. The antenna transmits pulses of radar energy into the pavement. These waves are reflected at significant layer interfaces and boundaries of dissimilar materials in the pavement. The reflected waves are captured by the system and displayed as a plot of reflection amplitude versus arrival time. The largest peak is the reflection from the pavement surface. The reflections of significance importance to engineers are those that occur after the surface echo. These represent significant interfaces within the pavement, and the measured travel time is related to the thickness of the layer. Apart from detecting the thickness of the surface layers from GPR data, dielectric values of surface and base layers are of particular interest as they indicate the presence of moisture and air voids in the pavement [5].

3.2.1.7 Acoustic Emission monitoring

Acoustic Emission (AE) refers to the generation of transient elastic waves during the rapid release of energy from localized sources within material. The source of these emissions in metals is closely associated with the dislocation movement accompanying plastic deformation and the initiation and extension of cracks in a structure under stress. Other sources of AE are: melting, phase transformation, thermal stresses, cool down cracking and stress build up [5].



The AE technique is based on the detection and conversion of these high frequency elastic waves to electrical signals. High-frequency acoustic energy is emitted by an object when it is undergoing stress, such as when corrosion products formed on a corroding rebar push out on the concrete surrounding it. The primary advantage acoustic emission monitoring offers over more conventional non-destructive evaluation techniques is that it results directly from the process of flaw growth [5]. AE technique is useful by means of detecting cracking, delamination, cleavage, and fretting in a material (damage accumulation as micro-crack) [6].

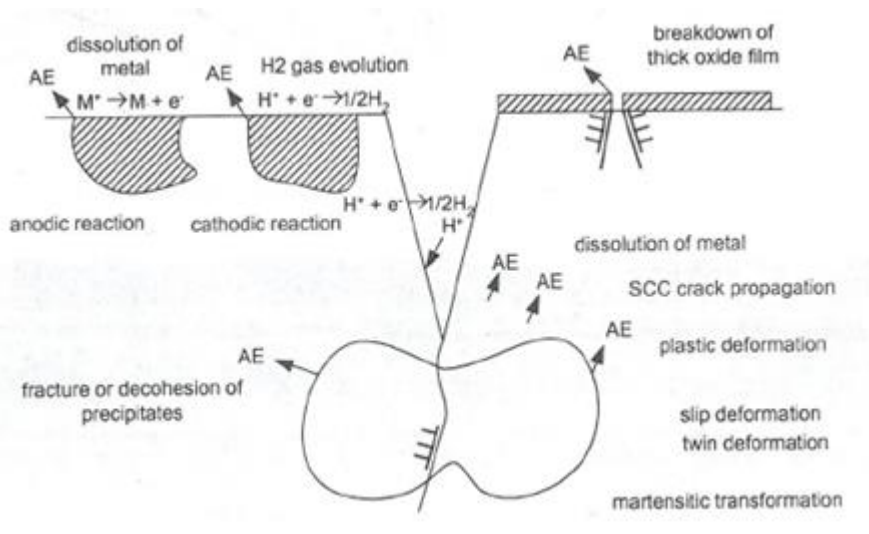


Figure 17 Schematic AE sources during corrosion, stress-corrosion cracking, and corrosion-fatigue process

Some acoustic emission sensors are:

- Wideband sensors
- High Hydrostatic Pressure Sensors
- Nuclear Radiation Resistant Sensors
- Variable Aperture Sensors
- Intrinsically Safe Sensors
- Miniature Sensors
- Water Tight and Underwater Sensors
- Rolling Sensors (dry contact)
- Airborne Sensors
- Unidirectional Sensors
- AE Integral Preamp Sensors

3.2.2 Reinforcement detection



Location and diameter of steel reinforcement can be determined destructively or non-destructively. As a result of the rapid development in non-destructive testing during the last decade, several reliable methods have been proposed. Nevertheless, under certain circumstances a destructive investigation may be more efficient. In order to minimize the damage to a structure it is advisable to search for statically important and expected reinforcement only. In the case of a beam it is recommended that the concrete along a small path on the underside is removed (see Figure 18). Number, diameter and concrete cover can be easily determined in this way. However, a possible second reinforcement layer in most practical cases will not be detected by using this technique [5].

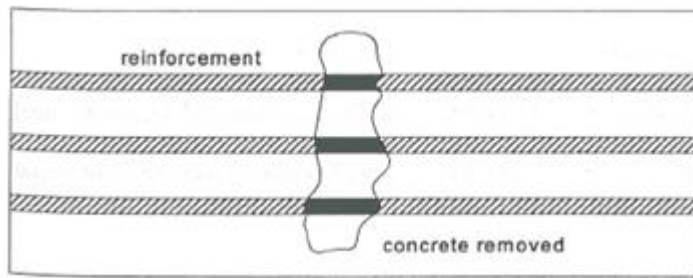


Figure 18 Bottom view of a reinforced concrete beam and removed concrete

For detecting the steel reinforcement of concrete slabs a combined destructive and non-destructive procedure is recommended. First the position of steel bars in both directions is determined. This can be done by using a simple electro-magnetic steel detector. Then, the concrete is removed in three spots, according to Figure 19. In this way it is possible to determine the steel diameters even if they are alternating [5].

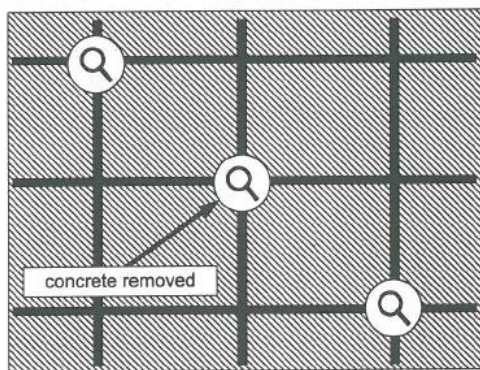


Figure 19 Bottom view of a concrete slab and location of the points for concrete to be removed

For the non-destructive determination of location and diameter of steel reinforcement three major groups of methods can be identified:

3.2.2.1 Electro-magnetic methods

Table 4 presents electro-magnetic methods for detecting and locating steel reinforcement [5].

Table 1. Electro-magnetic methods for detecting steel reinforcement



Physical effect	What can be determined?	Explanation
Permanent magnetism	Location, concrete cover	The attractive power between the reinforcement and a permanent magnet on the concrete surface is measured.
Electro-magnetic induction	Location, concrete cover, diameter	The magnetic flux is influenced by magnetic material in the electro-magnetic field.
Scattering of a magnetic field	Location, concrete cover, diameter	First the reinforcement is magnetized by a permanent magnet. Then the magnetic field is measured by using a hall probe. The steel reinforcement causes a scattering of the field.

The effect of magnetic induction is the one which is predominantly used in commercial devices. State-of-the-art products allow an easy scanning of the concrete surface and generate the results in an image format, see Figure 20 [5].

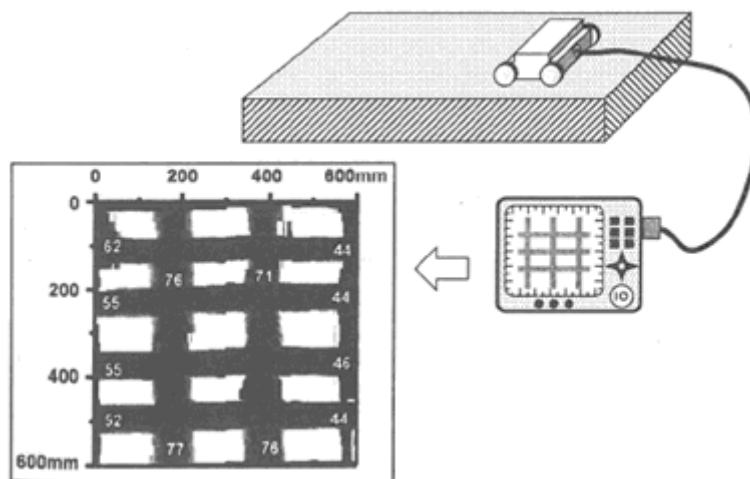


Figure 20 Electro-magnetic scanning of a concrete surface

The advantages of the electro-magnetic methods are [5]:

- The concrete cover can be determined reliably, whereas the determination of the bar diameters in practical cases sometimes causes problems.
- Results are obtained immediately. No time-consuming post-processing is necessary.
- State-of-the-art devices are cost-effective.

Some limitations of the electro-magnetic methods are [5]:

- The methods work reliably only up to concrete cover about 100 mm.



- For high reinforcement ratios the resolution of the method might be not sufficient. (Bars located close to each other are detected as one bar.)

3.2.2.2 Radiography

If gamma or x-rays penetrate a solid sample, a portion of the radiation passes the sample, a portion will be absorbed and another part will be scattered in other directions. The absorbed portion depends on the thickness and on the density of the sample as well as on the atomic number of the material. Because of the large difference in density between concrete and steel the absorption of gamma or x-rays can be used for detecting the steel reinforcement. If the radioactive source is located on one side of the concrete member and a photographic film on the opposite side the projection of the reinforcement will appear on the film, see Figure 21. If both the concrete cover and the bar diameter are unknown double exposure is a solution for determining the unknowns [5].

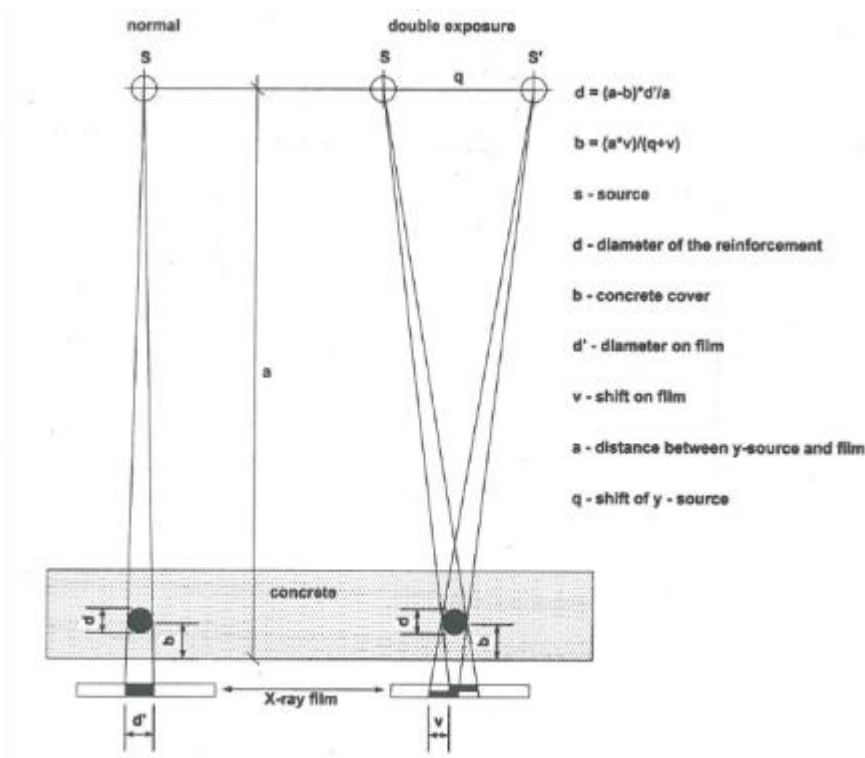


Figure 21 Schematics of radiography

For x-rays (200kV) the maximum concrete thickness is about 25cm and for Co-60 gamma radiation about 50cm. The use of linear accelerators allows even larger concrete thickness to be studied. Practical exposure times range from 3 to 20 min. If the thickness is more than the maximum thickness mentioned above, the source can be placed in a drill hole, see Figure 22. The same procedure is advisable if the structure is accessible from one side only. An additional effect of the drill hole radiography is an improvement of radiation protection [5].

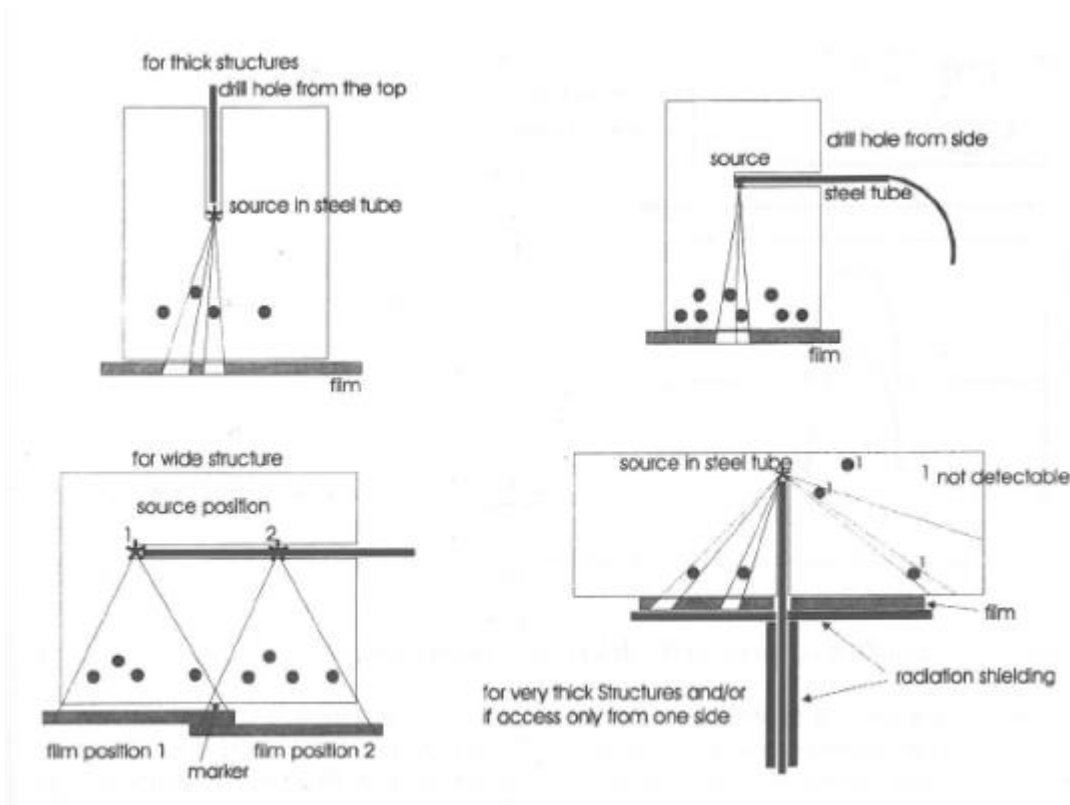


Figure 22 Drill hole radiography

The disadvantages of the radiography are the comparative high costs related to the work of specially trained staff and necessary radiation protection. In addition, no on-line information is provided because of the necessity of the film development. On the other hand, the results have an image format allowing an easy interpretation and documentation. This method is specially considered in the case of heavily reinforced parts of structures [5].

3.2.2.3 Radar methods

Electro-magnetic waves are reflected at interfaces between materials having different electrical properties. This effect can be used for detecting steel reinforcement in concrete structures, see Figure 23.

The advantage of the Radar principle is that the maximum inspection depth is about 50cm that means larger than for electro-magnetic methods. On the other hand, the interpretation of the data obtained appears to be difficult and reinforcement close to the concrete surface cannot be identified clearly. The Radar method, therefore, is beneficial in the case of large reinforcement diameters and thick concrete covers. A useful application is the location of pre-stressing cables. In this case the insensitivity against near surface reinforcement appears to be an advantage of this method [5].

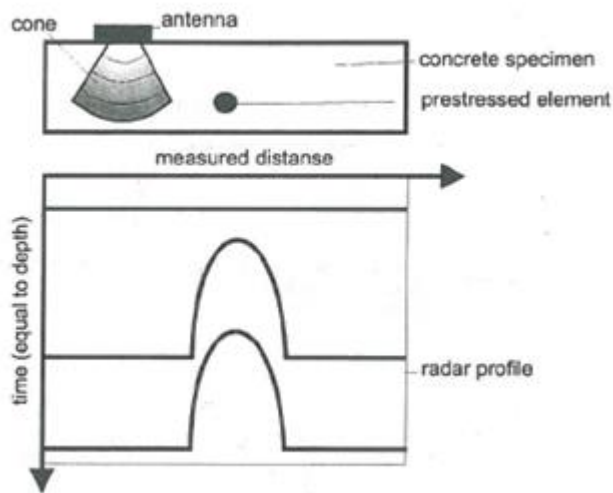


Figure 23 Radar method for detecting pre-stressed elements (Pöpel, Flohrer 1995)

3.2.3 Penetrant testing

This method is used to locate and identify surface defects in nonporous material. Further fields of applications are:

- Detection of cracking and porosity in welded joints
- Detection of surface defects in castings
- Detection of fatigue cracking in stressed materials

The surface of the part under evaluation is coated with a penetrant in which a visible or fluorescent dye is dissolved or suspended. The penetrant is pulled into surface defects by capillary action. After a waiting period to insure the dye has penetrated into the narrowest cracks, the excess penetrant is cleaned from the surface of the sample. A white powder, called developer, is then sprayed or dusted over the part. The developer lifts the penetrant out of the defect, and the dye stains the developer. Then by visual inspection under white or ultraviolet light, the visible or fluorescent dye indications, respectively, the defects are identified and located [5].

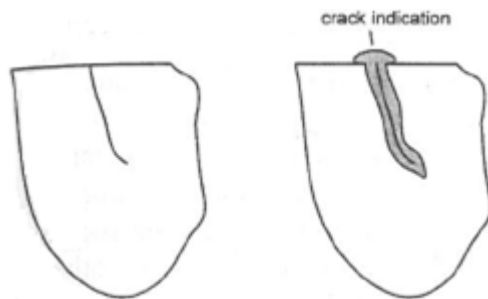


Figure 24 Liquid penetrant testing

3.2.4 Magnetic particle testing



This method is an NDT technique for crack identification that relies on local or complete magnetization of the component or surface being interrogated. It can only be applied to Ferromagnetic components. When a crack is present in the surface, then some magnetic flux will leak out from the sides of the crack (provided that the magnetic flow is in suitable direction relative to the crack) [5].

Figure 25 shows a magnetic field that is establish in a component made from ferromagnetic material. The magnetic lines of force or flux travel through the material and exit and reenter the materials at the poles. Defects such as cracks or voids are filled with air that cannot support as much flux, and force some of the flux outside of the part. Magnetic particles distributed over the component will be attracted to areas of flux leakage and produce a visible indication. If these particles are suitable colored, or the background is suitably colored, this concentration of particles will enhance the image of any cracks [5].

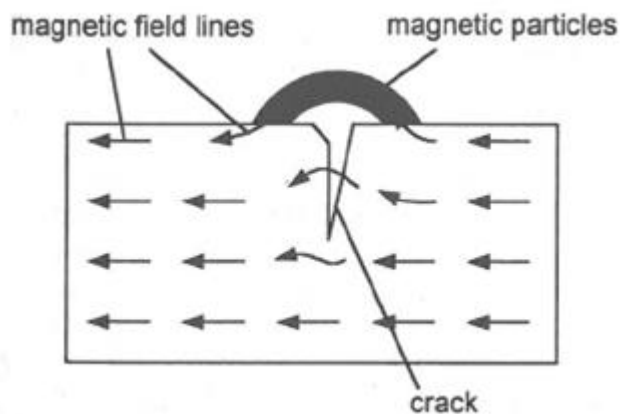


Figure 25 Magnetic particle testing

3.2.5 The Eddy Current Method

The eddy current has historically been used in the aerospace and power industries to test non-ferromagnetic cylinders. Its use has been expanded into civil engineering field to test the quality of welds and detect residual stresses in objects of any shape. The eddy current method involves placing an energized probe near the surface of the steel test component. If calibrated to the correct frequency, this will induce a current on the surface of the test component of a certain magnitude and phase. The eddy currents produced are proportional to the conductivity of the steel. When the eddy current passes over a crack or other discontinuity in the weld or steel component, it will cause a disruption in the current. The results can be instantly graphed on a handheld device to show the size and location of the discontinuity.

The eddy current method has several advantages that make it a practical choice for field inspections. The testing equipment consisting of a probe and data acquisition device is portable and available at a relatively low cost. The eddy current can penetrate both conductive and non-



conductive steel coatings, so that the coating system can remain intact during the inspection. Figure 26 shows a crack indication on a butt weld, represented by a large spiking area [10].

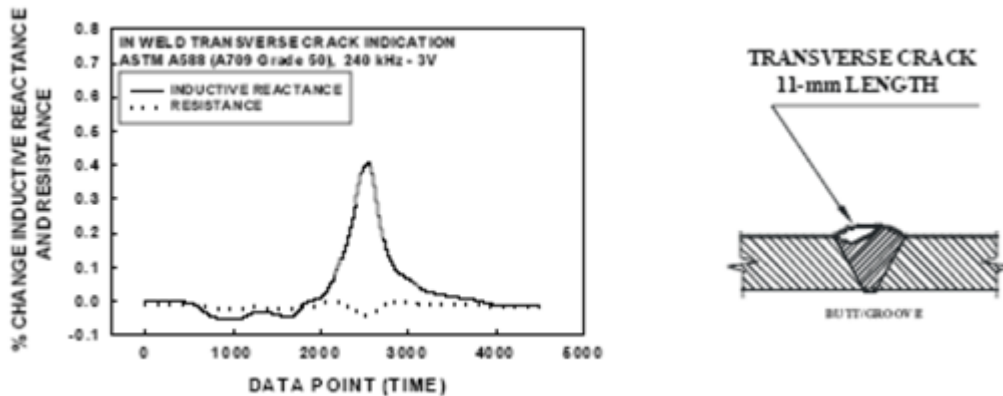


Figure 26 Example of crack indication on a butt weld, graphed from eddy current results [10]

There are several disadvantages associated with the eddy current method. Although the current can pass through coatings to detect defects in the steel underneath, the coatings do have a measurable effect on the test results. Since the effect is proportional to the coating thickness, this can be accounted for, but smaller defects may no longer be apparent when thicker coatings are used. It is also important that the probe be carefully calibrated before each inspection to ensure that the optimal frequency for the test metal is chosen. This leads to the need for a moderate amount of operator training and expertise [10].

3.2.6 The Radiographic Method

The radiographic method is used to inspect the quality of butt welds in the fabrication of steel plates for bridge girders. It works similar to an X-ray. Penetrating radiation is absorbed to produce a high contrast image. Indications of cracks and discontinuities in the welds will show up as darker areas on the high contrast image. Figure 27 shows an example of a radiograph, with the locations of two known cracks shown as dark horizontal lines [10].

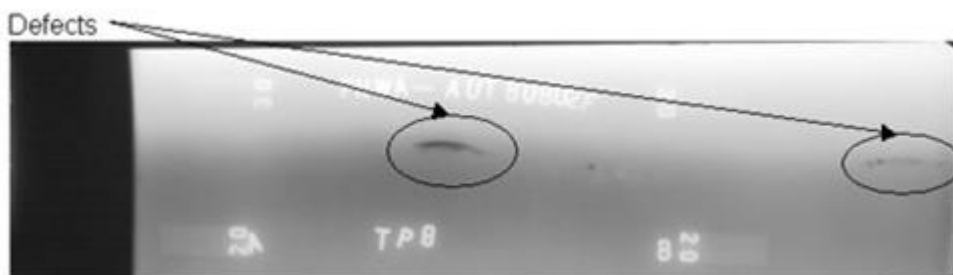


Figure 27 Typical radiograph image showing locations of two cracks [10]



The radiograph produces a two dimensional image that can be subjectively interpreted to determine the locations and size of cracks. The radiograph also provides a permanent record of the inspection for future reference. One of the disadvantages involved in radiographic method is that it poses a health hazard due to the radiation exposure. This results in increased setup and inspection time to erect barriers to limit exposure and ensure proper safety precautions are taken. Also, the radiograph produces a two dimensional image, so it is not capable of determining the depth of cracks and discontinuities, only their size and location [10].

3.2.7 The Ultrasonic Method – Manual vs. Automated

The ultrasonic method works by measuring the trip time of ultrasonic pulses emitted by a transducer traveling through a test component. The test system is composed of three main components. The ultrasonic pulse echo transducer emits the ultrasonic pulse of known velocity and frequency through the test component. A computerized data acquisition system collects the data from the ultrasonic pulse. Lastly, a spatial control system tracks the coordinates of the transducer as it moves along the surface of the test component, so that indications of discontinuities can be located on a coordinate system.

Variations in the pulse velocity will show indications of cracks and discontinuities in the test component. Data from the data acquisition system can be plotted on a three dimensional image that shows crack size, location, and depth, which gives it an advantage over the two dimensional image from the radiograph. Initially, the ultrasonic transducer was only operated manually. This required the operator to calibrate the frequency of the transducer while simultaneously moving it along the test location, leaving room for human error in the inspection process.

Recent advances in computer technology have led to development of automated ultrasonic testing. Automated ultrasonic testing uses a robotic arm with a wide range of motion to move the transducer around the test location. This ensures complete coverage of the area under inspection, and minimizes human interaction during the test [10].

Figure 28 shows a sample P-Scan image amplitude profile displayed on logarithmic scale, and Figure 29 shows the same image displayed on a linear scale. The threshold line shown in the figure is selected based on plate thickness and acceptance/rejection criteria in the AASHTO Bridge Welding Code. Amplitudes exceeding this threshold are shown in colors representing the decibel level and are indications of potential unacceptable defects [10].

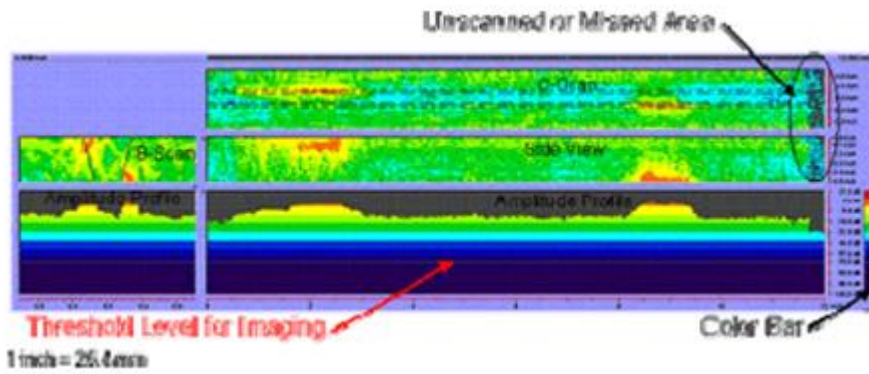


Figure 28 P-scan image displayed on logarithmic scale [10]

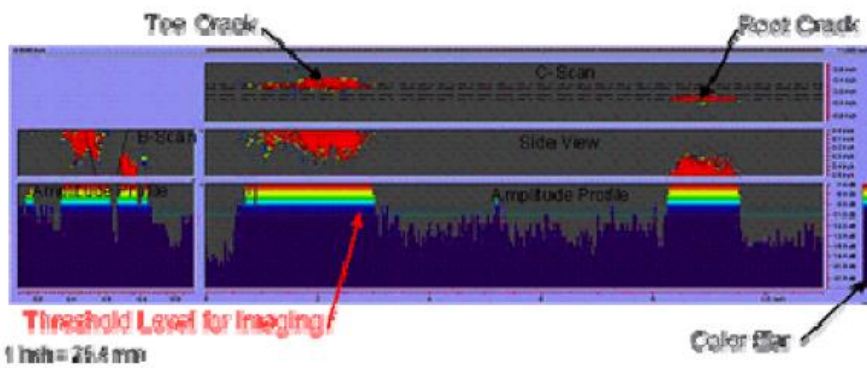


Figure 29 P-scan image displayed on linear scale [10]

Ultrasonic testing in general poses less of a health risk than the radiograph method, and utilizing the automated ultrasonic method provides more objective results by minimizing human interaction. However, the automated ultrasonic method requires longer setup, calibration, and inspection time when compared to the radiographic and manual ultrasonic method. The equipment is also bulkier than the other methods, making it difficult to transport between testing sites. Although the actual test is automated, operators must still be trained in proper calibration of the equipment. Also, it is a relatively new technology, so more research is needed before the full extent of its applications in bridge inspection is known [10].



4 References

- [1] Wang, T., L., Zong, Z., 2002, Department of Civil & Environmental Engineering – Florida International University, Miami, Research Report No. FL/DOT/RMC/6672-818 “Improvement of evaluation method for existing highway bridges”
- [2] Worden, K., Farrar, C. R., Manson, G., Park, G., Proceedings The Royal Society; Downloaded from rspa.royalsocietypublishing.org, July 29, 2011, “The fundamental axioms of structural health monitoring”
- [3] Sohn, H., Farrar, C. R., Hemez, F. M., Shunk, D. D., Stinemates, D. W. Nadler. B. R., Czarniecki, J. J., 2004, Los Alamos National Laboratory Report, LA-13976-MS, A Review of Structural Health Monitoring Literature: 1996–2001.
- [4] Maksymowicz, M., University of Minho, Portugal; P.J.S. Cruz, University of Minho, Portugal; J. Bień, Wroclaw University of Technology, Poland; R. Helmerich, Federal Institute for Materials Research and Testing, Germany; International Association for Bridge Maintenance and Safety, Concrete railway bridges – taxonomy of degradation mechanisms and damages identified by NDT methods.
- [5] Fib CEB-FIP, Bulletin No. 22, State of art report, March 2003, “Monitoring and safety evaluation of existing concrete structures”
- [6] RILEM Technical Committee (Masayasu Ohtsu), Materials and Structures, Volume 43, Issue 9, November 2010, “Recommendation of RILEM TC 212-ACD: acoustic emission and related NDE techniques for crack detection and damage evaluation in concrete. Measurement method for acoustic emission signals in concrete”
- [7] Whitlock Dalrymple Poston & Associates, Inc. 2011, www.wdpa.com
- [8] Chen, B., Department of Mechanical Engineering—Engineering Mechanics, Department of Electrical and Computer Engineering, Michigan Technological University, Houghton, MI, USA, Liu, W., Department of Electrical and Computer Engineering, Michigan Technological University, Houghton, MI, USA, 2010, Computer-Aided Civil and Infrastructure Engineering 25, 504–516, Mobile Agent Computing Paradigm for Building a Flexible Structural Health Monitoring Sensor Network.
- [9] NDHRP (National Cooperative Highway Research Program), Report 381; Washington, D.C. 1996, “Report on the 1995 Scanning Review of European Bridge Structures”
- [10] Bader, J., 2008, Nondestructive Testing and Evaluation of Steel Bridges, ENCE 710.

D 2.1 – Appendix 4

Repair and strengthening of concrete bridges





D 2.1 – Appendix 4

Repair and strengthening of concrete bridges

Version 2, 20 February 2017

Colophon

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1 Introduction

The strengthening or retrofitting of existing concrete structures to resist higher design loads, correct deterioration related damage, or increase ductility has traditionally been accomplished using conventional materials and construction techniques. Externally bonded steel plates, steel or concrete jackets and external post-tensioning are just some of the many traditional available techniques. Concrete bridge repair can be difficult and costly. Especially when the road and lanes must be closed, that results in traffic disruption and delays. Another important aspect is to keep clearance beneath the bridge, especially when it crosses a trafficked waterway, railway or other section of highway.

Composite materials made of fibers in a polymeric resin, also known as Fiber Reinforced Polymers (FRP), have emerged as an alternative to traditional materials and techniques. FRP materials are lightweight, noncorrosive, and exhibit high tensile strength. Additionally, these materials are readily available in several forms ranging from factory-made laminates to dry fiber sheets that can be wrapped and conform the geometry of a structure before adding the polymer resin. The growing interest in FRP systems for strengthening and retrofitting can be attributed to many factors. Although the fibers and resins used in FRP systems are relatively expensive compared to traditional strengthening materials like concrete and steel, labor and equipment costs to install FRP systems are often lower. FRP systems can also be used in areas with limited access where traditional techniques would be difficult to implement.

Repair and strengthening of concrete structures is a complicated and complex issue. It is effective only if the detailed bridge assessment is performed. Plan for repair and/or strengthening of the structure should be based on:

- Damage detection (determination of the reasons and results),
- Serviceability requirements,
- Construction requirements.

At the planning stage, the aim of the strengthening and repair need to be determined. The aim can be:

- Protection of structure (e.g. reinforcement)
- Aesthetic
- Bringing of the damage element to the participation in load transfer process and its performance as it is primary designed.

Overloading due to increase in wheel loads and regular exposure to aggressive external environment may aggravate the situation further. Post-tensioned concrete bridges may also exhibit loss in pre-stress over the time, resulting in reduction in load carrying capacities of the affected



members. Poor quality of construction and lack of regular maintenance could potentially lead to major retrofit in a bridge structure. Furthermore, components facilitating expansion/contraction of bridge and load transfer to the sub-structure, such as expansion joints and bearings, may also require rehabilitation or replacement over time. Defects in the constituent materials may be manifested in the form of cracking/spalling of concrete, excessive deflection of structure, corrosion of steel components/reinforcement etc. It is evident that rehabilitation of bridges involves addressing many problems and no single technique or retrofit method could offer a complete solution and answer for all needs.

In repairing deteriorated concrete bridges, the primary objective is to restore the structure to its original shape and condition by using a material that will ensure structural integrity, durability, and composite behavior, while matching the existing concrete in color and appearance. The repair material should be at least as strong and durable as the existing concrete. Physical and chemical properties such as modulus of elasticity and coefficient of expansion should be similar to the existing concrete. It is essential that the repair material should not shrink; otherwise shrinkage cracks will develop.



2 Conventional bridge strengthening and repair techniques

2.1 Repair by injecting cracks and voids

Cracks can lead to one or two of the following consequences:

- An immediate negative influence on the structure's load bearing capacity
- A negative influence on the durability

If the crack in the structure has negative influence on its load bearing capacity, it is necessary to connect disjoined elements, irrespective of crack width. The load carrying capacity needs to be higher than the capacity of separate elements.

When the cracks cause only leakiness of element or structure, the surface cover is performed (for the crack up to 0.2 mm) or the crack is filled with sealer. An important issue that needs to be clarified is if the crack width changes with load or temperature conditions (active crack) or if it is constant (passive crack).

In general cracks are filled with the injection method. Injections are divided into two types:

(1) Under gravity

(2) Under pressure, divided into:

- Low-pressure injection, i.e. when the pressure does not exceed 0.8 MPa; it is applied to cracks in a low quality concrete and to cracks with large openings,
- Medium-pressure injection, i.e. the pressure varies between 0.8 – 8.0 MPa; it is applied to cracks ≥ 0.5 mm, cracks in pre-stressed concrete or concrete with high density of reinforcement,
- High-pressure injection, i.e. when the pressure exceed 8.0 MPa; it is applied for cracks with small opening (0.1 – 0.3 mm) in high strength concrete.

There are three main categories of resins used for sealing and filling of cracks and voids:

(1) Epoxy resins

Epoxy resins are applied mainly to *bond* cracked elements. The element which cracks and is filled with epoxy resin can take active part in the load transfer. Epoxy resins are characterized by low viscosity (100 – 500 MPa·s), do not contain thinners or only active thinners. An important issue that can strongly influence binding of epoxy resins is humidity. Water can have negative impact, deactivate hardener and also weaken the joint. The manufacturer's recommendations for the epoxy adhesive shall be strictly followed as to the requirements for safety precautions in handling the epoxy, storage of the material, mix proportions of the two components and application temperatures.

(2) Polyurethane resins



Polyurethane resins due to their mechanical properties (especially high elastic deformation) are applied generally to *seal* the structure. During contact with water polyurethane resins binds and rapidly undergo foaming, this results in creation of elastic mass.

(3) Acrylic resins

Another type of resins that are used for *sealing* is acrylic resins. They are characterized during hardened state by high compression strength (~70 MPa) and tensile strength (more than 30 MPa); moreover they have very good concrete adhesion. In comparison with epoxy and polyurethane composites, acrylic resins have lower viscosity, which decreases with the decrease of temperature.

For large cracks and seems, cement with added polymers for plasticity and swelling characteristics can also be used.

2.2 Repair by adding repair material

When concrete elements are superficially damaged (mechanical damage, de-icing damage or chemical damage such e.g. sulphates) repair of the outer layer of concrete could be sufficient. When concrete is damaged due to corrosion of the reinforcement bars, one should check the cause of corrosion (carbonization and/or chloride) and additional repair of reinforcement bars could be needed.

Conventional repair of concrete covers:

- Surface preparation
- Repair of reinforcement bars (if needed)
- Applying repair material

2.2.1 Surface Preparation

Surface preparation is needed for obtaining a good bond between the repair material and the original concrete.

Surface preparation covers:

- Removal of weak concrete layers and cement slurry,
- Removal of concrete cover around corroded reinforcing bars and cleaning of visible reinforcement,
- Cleaning of the concrete surface from dust, water, sand, etc.
- Applying primer on the cleared surface (optional).

The depth to which concrete has to be removed is depending on the cause of damage. If not enough concrete is removed (e.g. layers of concrete containing chlorides around reinforcement bars)



corrosion process will continue. Also, contamination from contaminated concrete can migrate to the repair material and provoke contamination of this new material. It is highly important that on the edge between old and new material, corrosion process would not be activate. Therefore the maximum content of chlorides in adjacent layer should not exceed 0.2%. It should be assured that the concrete surface has an adhesive strength of at least 1.5 MPa.

For removal of concrete, the following techniques can be used:

- By hammering; this is used for small scale concrete repair
- By milling/bushing; with this technique, up to 10 mm can be removed (Figure 1)
- By abrasive blasting; with these techniques, large surfaces can be removed using ice, water or grit as abrasive.
- Pneumatic/electric; pneumatic or electric drills are used when in depth concrete removal is needed.
- High pressure abrasive water blasting; this technique is used when in-depth concrete removal is needed (Figure 2)



Figure 1 Bushing (source: portlandonline.com)



Figure 2 High pressure water blasting (source: APA concrete repairs Ltd.)

The type of surface preparation for reinforcement bars is dependent on the cause of corrosion and type of repair material and can vary from doing nothing, to cleaning, to priming/coating the reinforcement bars. For cleaning reinforcement bars, one can use a wire brush or abrasive blasting. Surface coating application on reinforcement bars strongly depends on the cause of damage and type of the repair material. Surface coating is based on the coating of reinforcement bars before pouring concrete with anticorrosion layers like: zinc, epoxy resins, epoxy paintings, PVC coatings. For the coating with zinc, effectiveness of the process strongly depends on many factors like: pH of concrete, chemical concrete composition, content of chlorides (with a chloride content more than 1%, zinc coatings damage fast and this protection method should be treated as a temporary). Zinc and zinc coatings are affected by corrosion process in concrete. In the first phase the corrosion



progresses is very fast and then slows down due to the creation of waterproof layers. During first days loss of zinc is around 5-10 μ m. After this time the corrosion process rapidly decays.

The steel surface should be also cleaned before coating application. For repair of structures, resins or mineral binders are very common. Execution of repairs with use of coating materials based on epoxy resin is possible when the surface temperature is more than 8°C and is higher by 3°C than dew point.

The advantages of coatings with mineral binders are:

- Possibility of application in temperature more than +5°C,
- Short work breaks among work cycles,
- Independency of working temperature from the dew point,
- Possibility of application on humid steel bars,
- Less stringent requirements of cleanness of steel bars,
- Cost efficiency.

2.2.2 Repair of reinforcement bars (if needed)

If the diameter of reinforcement bars is significantly narrowed down due to corrosion, extra reinforcement should be added.

2.2.3 Applying repair material

Currently in use are two methods for applying repair material to concrete: casting and shotcrete. Loss filling by mortar or Polymer Cement Concrete are the most popular methods for concrete repair.

2.2.3.1 Casting

With casting, a casting mould is applied to the structure and repair material is poured. After hardening, the mould can be removed.

For repair of bridges using in-situ casting technology, different materials can be utilized:

- Cement Concrete (CC)
- Polymer Cement Concrete (PCC)
- Polymer Concrete (PC)
- Fiber-concrete (concrete reinforced by steel, carbon, polymer, fibers, etc.)

CC is conventional concrete. Often, a batch of concrete monomer or polymer emulsion is added to CC to improve physical parameters, which is then called PCC. PCC is the most cost-efficient and therefore it is recommended for repair or strengthening of large areas of the structure.

PC is a composite without cement that is created from the mix of aggregate with a liquid monomer that takes a role of binder. Technology of PC is similar to traditional concrete and can be applied



with the use of the same equipment (with additional use of drying equipment – if necessary). Advantage of CC and PCC is that a material of high pH is poured around the reinforcement bars, so the passivation layer will restore.

PC can be used for superficial damage and to repair small parts. It is not suitable as a repair material for large parts due to shrinkage and low load bearing capacity. It is not suitable as a repair material when PC cannot be casted in temperature higher than 8°C and if the surface and air temperature are not higher by 3°C from the dew point. Important limitation in application of PC is the high thermal expansion and vulnerability to changing humidity (of air and surface) at low temperatures. They have also limited fire resistance.

Fiber-concrete is a composite material that consists of short and thin fibers (steel, carbon, aramid or synthetic) added to the cement mortar or to small grain concrete. In general, fibers have thickness between 0.2-0.5 mm, and length 20-50 mm. Added fibers limit cracking including shrinkage cracks, increase tensile strength and resistance for dynamic load. Synthetic fibers are non-reactive during chemical synthesis and are resistant to alkali.

2.2.3.2 Shotcrete (*Sprayed concrete*)

Shotcrete is a construction technique of projecting concrete on a given surface (Figure 3). The idea is to produce dry mix of sand and cement out of the application place and then transport it to the casting place through a hose and to pneumatically project at high velocity onto a surface. Depending on the type of concrete mix, the method is divided into: dry or wet (comparison is shown in Table 1). Shotcrete is used in retrofitting of bridge structures for repair of corroded concrete surface, repair of lacks of concrete, casting of concrete cover for additional reinforcement added for strengthening of the structure, surface concrete protection. Minimal thickness of the sprayed concrete is 2 cm. For the shotcrete material, cement concrete or concrete modified with resin are used. In order to increase physical parameters of concrete, different kinds of fibers (steel, carbon, polypropylene, etc.) are added to the concrete mix.



Figure 3 Shotcrete (wet) (source: VTG Beton)

Table 1 Comparison of dry and wet shotcrete method

Dry shotcrete method
Very good density of concrete mixed caused by dynamic convey, it is comparative with the density obtained with the best vibration methods in traditional pouring of concrete
Law water/cement ratio i.e. 0,35-0,50; this contributes to creation of compact and hard layer and allows to form thick concrete layers with good mechanical and service properties
Favorable concrete structure provoked by the way of placing the concrete on the structure
Possibility of transporting of concrete mix to 500 m horizontally and to 200 m vertically
Possibility of changing of mix-consistency depending on the atmospheric conditions and types of damages of the structure
Lower weight of equipment and better mobility in comparison with wet method
High resistance for cracking
<i>This method is recommended for repair and strengthening of bridge infrastructure especially deep loses of concrete</i>
Wet shotcrete method
Lower quantity of grains of aggregate that are rebound in first phase of shotcrete process
Lack or random occurrence of cement shotcrete layers close to surface
Higher homogeneity and agreement of mechanical properties of concrete with design assumption
Lack of dust on the working place
Lower density of concrete in comparison with dry method
<i>Wet method is recommended for tunnels</i>

2.2.3.3 Sprayed Polymer Cement Concrete



Due to stringent requirements on the components used for shotcrete, SPCC (Sprayed Polymer Cement Concrete) with polymer-cement binder is usually used. These materials are characterized by high compressive strength, low shrinkage properties and very good surface adherence. SPCC has also very good resistance for de-icing chemicals applied on the road during winter. They require less accuracy in curing process due to lower thickness. The resin cannot activate and start binding before application of the shotcrete on the surface.

2.3 Cathodic protection as a preventive measure

Concrete is a porous material, which readily absorbs contaminants from the surrounding environment. The water and oxygen in presence of chlorides react with iron and creates corrosion products on the surface of the reinforcing steel. The increase due to larger volume of the corrosion products exerts tensile stress that eventually causes the concrete to crack and spall as illustrated in Figure 4.



Figure 4 Cracking and spalling of the concrete due to internal reinforcement corrosion

2.3.1 Corrosion process

Corrosion of the reinforcement steel is the main cause of deterioration in RC structures which is often triggered by chloride attack or carbonation. The reinforcement in the concrete is protected against corrosion by a very thin passive layer as long as the concrete remains chloride free and the alkalinity is sufficient ($\text{pH} > 12$). The passive layer can be destroyed by chloride contaminated gravel or sand, concrete additives or the penetration of de-icing salt that finds its way through pores and cracks in the concrete until it exceeds a critical concentration. The metal loss represents the anodic reaction (see Figure 5).

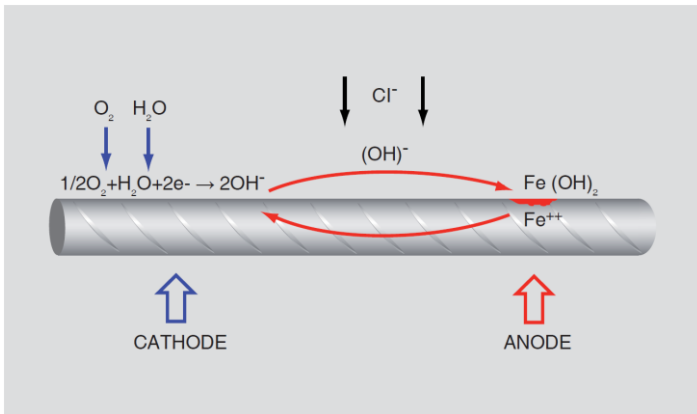
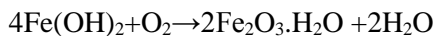
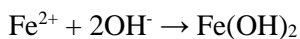
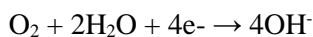
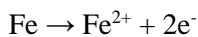


Figure 5 Local cell corrosion

The corrosion of iron takes place through a process called anodic reaction. In the presence of water containing dissolved oxygen, the corrosion can take place explained by the following equations:



2.3.2 Cathodic protection

Cathodic protection is a technique used to control the corrosion of a metal surface by making it the cathode of an electrochemical cell. In other words, the principle is connecting an external anode to metal to be protected and passing of an electric DC current so that all areas of the metal surface become cathode and do not corrode. The method relies on the relationship between the potential of steel and the corrosion rate. The potential of the reinforcement is brought to a stable passive state through installation of a negative protective current. Through the formation of hydroxide ions on the reinforcement the protective passive layer is restored and thus the corrosion is prevented.

Cathodic corrosion protection is considered as a preservation as well as preventive corrosion protection method which is used in a wide field of civil engineering applications and mostly in developed countries. The application of cathodic corrosion protection makes sense and is beneficial when the damage in the reinforcement and concrete is not too severe. Cathodic protection, in certain cases, is an attractive alternative and/or supplement to conventional repair works. The idea of using cathodic corrosion protection for the rehabilitation of steel reinforced concrete constructions was established in the mid-1970s in the USA. In Europe, it was in UK where the first attempts to employ this technique for corrosion prevention in reinforced concrete constructions were made.



Cathodic corrosion protection is considered as an active protection method which solves the corrosion problem by eliminating its cause. Cathodic protection intervenes into the electrochemical corrosion process. Through application of an anode system on the concrete surface a protective current is opposed to the corrosion current. This protective current polarizes the reinforcing steel in a way that the steel cannot corrode anymore. The bare reinforcement, which is accessible at some points, is connected to the minus pole and the anode to the plus pole of a rectifier which serves as current source. The reinforcement is polarized through electron flow in a way that it is re-passivated. Cathodic protection is achieved by either (1) sacrificial anode (e.g. zinc, magnesium) or (2) impressed current method. While sacrificial anode protection systems are simple and cheap to install, reliable and free from operator surveillance, the impressed current method is more flexible under varying operation conditions, it presents a lower LCC and large structures can be protected using this method. It is the impressed current method which is more common in RC structures as illustrated in Figure 6.

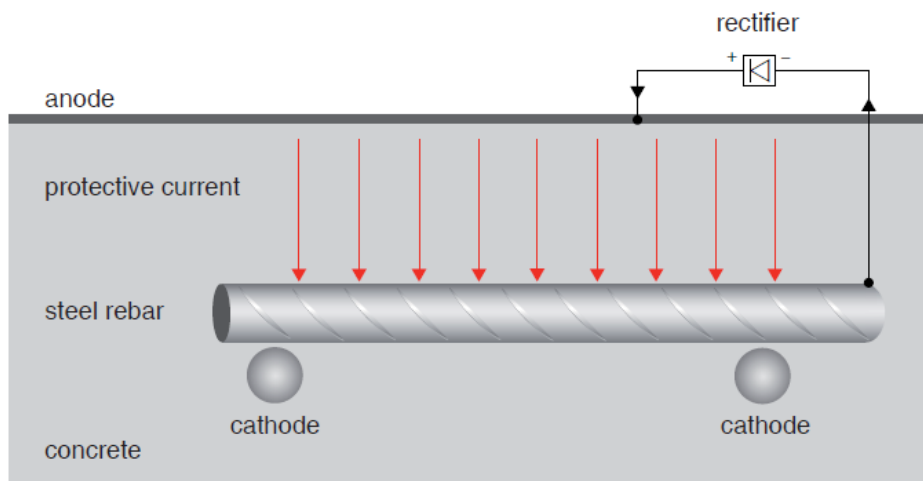


Figure 6 Cathodic protection of internal reinforcement in reinforced concrete

2.4 Strengthening using externally bonded steel plates

A common method for strengthening of concrete bridges is bonding of steel plates, see Figure 7. External steel plates are applied in areas where the tensile strength is insufficient because of a deficiency of the reinforcement necessary to carry the load. Application of bonded steel plates can increase the flexural strength and load carrying capacity. The results are improved stiffness (reduction of deflection and cracking) and enhanced shear capacity. The resistance of the adhesive in case of fire should be additionally analyzed, if there is a risk of fire. The process is not designed to carry dead load, but only live loads. The reinforcement is normally in form of steel plates. Bolts are often used in conjunction with adhesive to provide a mechanical anchor for the plates at the



ends to prevent premature debonding due to peeling, and also to help supporting the plates whilst the adhesive cures. Plates can be placed in areas where either the bending, shear or compression capacity needs to be increased. In bending the plate acts as an extra layer of tensile bending reinforcement. The tensile forces have to be transferred from the concrete to the steel plate, and it is the function of the adhesive to transfer the tensile forces by means of shear stresses between the steel plate and the concrete surface and to produce a continuous bond to ensure that full composite action is achieved.

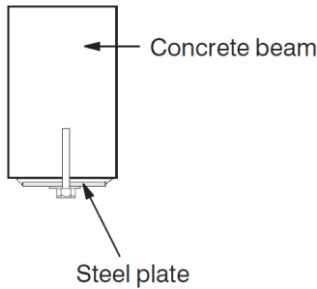


Figure 7 Scheme of an externally bonded steel plate to a concrete beam

In general, this technique is useful for the flexural and shear strengthening of bending elements, such as beams and slabs, and for the compression capacity enhancement of columns. Steel plates are hard to lift and need to be tailor-made to suit to the as-built dimensions of the members. Resulting surface finish is unsightly and steel plate retrofit is prone to corrosion over time.

Externally bonded steel jackets are good solution for strengthening of concrete columns. This technique can be chosen when the loads applied to the column will be increased, and at the same time, increasing the cross sectional area of the column is not permitted. First step is to remove damaged concrete cover, to clean reinforcement (e.g. with wire brush or sand blasting) and to coat the bars with anti-corrosion material (if needed). Then the steel jacket needs to be installed (size and thickness based on the design) and through openings CC or PCC material is poured. In that way, sufficient bonding between concrete column and steel jacket is guaranteed.

2.5 Strengthening using external post-tensioned tendons

External post-tensioning is often used for strengthening of load deficient and damaged bridge structures. External post-tensioning can be achieved by employing pre-stressing strands, high strength bar tendons, steel plates or other materials. Post-tensioning process can be seen in Figure 8. Over the service life of a pre-stressed concrete member, loss in pre-stress may occur due to a variety of reasons. Post-tensioned bridges can be effectively rehabilitated by external post-tensioning technique to compensate for loss in pre-stress or increase the flexural load carrying capacity. In this technique, pre-stressing tendons or bars are located according to pre-determined profile on the external surface of the member to be strengthened according to design. Anchor heads are positioned at the ends of these tendons/bars to post-tension the member using hydraulic jacks.



Although, this method is quite effective, but requires sufficient strength in the existing concrete to transfer the stress. Furthermore, exposed tendons and anchorages need to be protected against corrosion and vandalism.



Figure 8 External post-tensioning [1]

2.6 Strengthening with near-surface mounted reinforcement

Near-surface mounted (NSM) technique has been used in Europe for strengthening of concrete structure since early 50s. First application of NSM strengthening was in Sweden and was related to concrete bridges. NSM is accomplished by grooving the surface, filling the grooves with cement mortar, or a suitable adhesive material, and embedding steel rebar (or FRP reinforcement) in them. NSM technique can be applied for the beams, slabs and bridge columns.

2.7 Strengthening with increasing reinforcement and cross-sectional area

When concrete beam does not require a great deal of increase in load bearing capacity, it is enough to increase the amount of reinforcement by adding extra bars. Additional reinforcement can be welded to old one after removal of concrete cover. Extra reinforcement should be poured with cement mortar or shotcrete can be used (the height of the beam increase by 2-8 cm), see Figure 9.



When concrete beam requires a significant increase of the bearing capacity, the strengthening can be performed by increasing the amount of reinforcement and the cross-sectional area of the section as illustrated in Figure 10

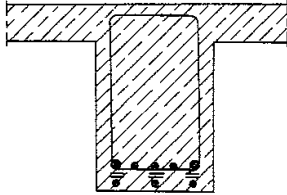


Figure 9 Example of a beam with added internally reinforcement

If the bending or shear resistance of the beam needs to be significantly increased or the beam is strongly damaged, reinforced concrete mount should be introduced, Figure 7.

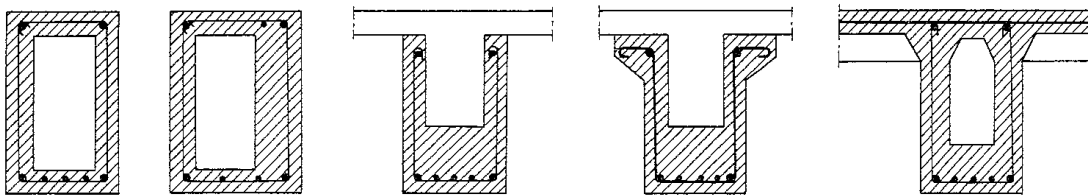


Figure 10 Example of different sections with additional reinforced and increased cross-section

Beams with diagonal cracks that appeared due to high shear stresses should be reinforced with steel wrap. The reinforcement should be covered with steel mesh and cement mortar or shotcrete need to be applied, Figure 11.

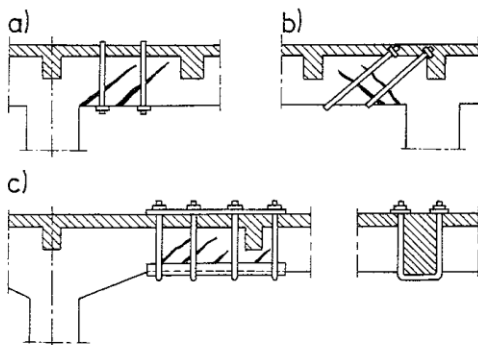


Figure 11 Example of a beam with additional shear reinforcement

2.8 Concrete jacking

Jacketing method with reinforced concrete is widely used strengthening method for concrete columns, Figure 12. The size of the jacket and the number and diameter of the steel bars used in the jacketing process depend on the structural analysis that was made for the column. If the load capacity of the column needs to be significantly increased, first step is to remove concrete cover, to



clean the old reinforcement and then to coat the bars with material (if needed) that would prevent corrosion. The jacketing process starts by adding steel connectors into existing column in order to fasten new stirrups and bars, after this the holes need to be filled with appropriate component (e.g. CC or PCC). Next step is to pour the concrete of the jacket.

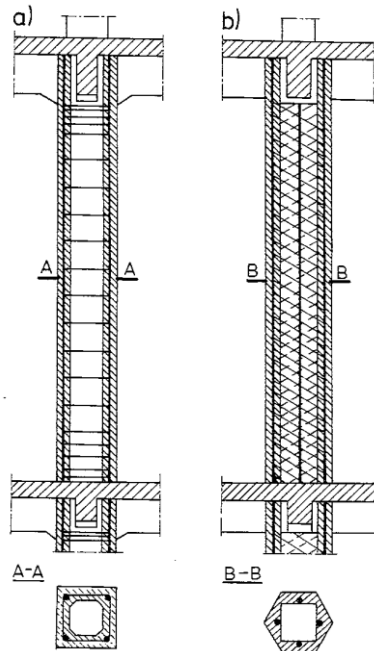


Figure 12 Example of reinforced concrete jacket for a column

2.9 Strengthening by change in static system

Usually effort of different elements of a bridge is not the same for all of them. For this reason, the change of static system may increase capacity of the bridge. The strengthening of the concrete bridges can consist of the following operations [3]:

- providing structural continuity to simple span bridges,
- elimination of intermediate hinges in spans,
- change of beam system to frame system,
- change of frame system with cantilever arms to frame system,
- change of beam or frame systems to cable stayed systems.

Most frequently, the first option is implemented. This is common in particular beam bridges made of prefabricated elements – Figure 13. After establishing the continuity, beam can be rested on one or two bearings.

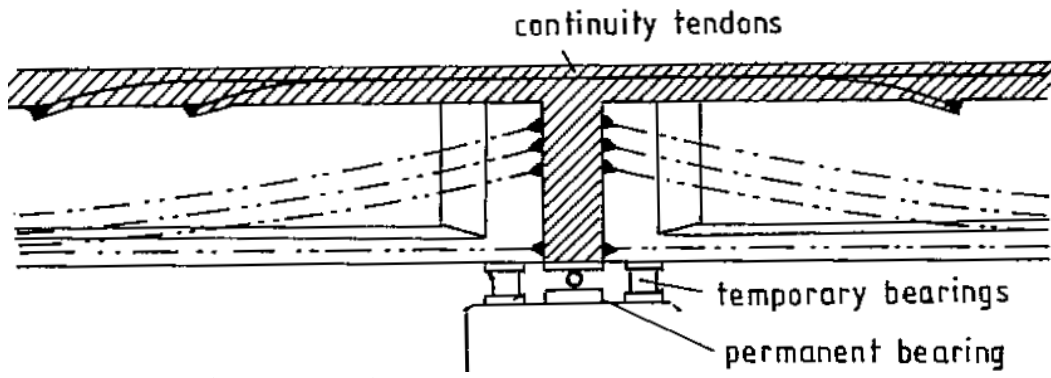


Figure 13 Providing structural continuity [3]



3 Externally bonded FRP systems

3.1 Types of FRP strengthening systems

The FRP products can be divided into three main types:

- (1) **Wet layup systems:** consist of dry unidirectional or multidirectional fiber sheets or fabrics impregnated with a saturating resin on-site. The saturating resin, along with the compatible primer and putty, is used to bond the FRP sheets to the concrete surface. Wet layup systems are saturated in-place and cured in-place and, in this sense, are analogous to cast-in-place concrete. Three common types of wet layup systems are listed as following:
 - Dry unidirectional fiber sheets where the fibers run predominantly in one planar direction;
 - Dry multidirectional fiber sheets or fabrics where the fibers are oriented in at least two planar directions; and
 - Dry fiber tows that are wound or otherwise mechanically applied to the concrete surface. The dry fiber tows are impregnated with resin on-site during the winding operation.

Wet layup process is shown in Figure 14.



Figure 14 Wet layup process (Mostostal Warszawa S.A.)

- (2) **Prepreg systems:** consist of uncured unidirectional or multidirectional fiber sheets or fabrics that are pre-impregnated with a saturating resin in the manufacturer's facility. Prepreg systems are bonded to the concrete surface with or without an additional resin



application (Figure 15), depending upon specific system requirements. Prepreg systems are saturated off-site and, like wet layup systems, cured in place. Prepreg systems usually require additional heating for curing. Prepreg system manufacturers should be consulted for storage and shelf-life recommendations and curing procedures. Common types of Prepreg are:

- Pre-impregnated unidirectional fiber sheets where the fibers run predominantly in one planar direction;
- Pre-impregnated multidirectional fiber sheets or fabrics where the fibers are oriented in at least two planar directions; and
- Pre-impregnated fiber tows that are wound or otherwise mechanically applied to the concrete surface.



Figure 15 Application of the Prepreg based on glass fibers in order to increase shear capacity of the beam

(3) **Pre-cured systems:** consist of a wide variety of composite shapes manufactured off-site. Typically, an adhesive along with primer and putty is used to bond the pre-cured shapes to concrete surface. The system manufacturer should be consulted for recommended installation procedures. Pre-cured systems are analogous to precast concrete. Common types of pre-cured systems are:

- Pre-cured unidirectional laminate sheets, typically delivered to the site in the form of large flat stock or as thin ribbon strips coiled on a roll;
- Pre-cured multidirectional grids, typically delivered to the site coiled on a roll;
- Pre-cured shells, typically delivered to the site in the form of shell segments cut longitudinally so they can be opened and fitted around columns or other members; multiple shell layers are bonded to the concrete and to each other to provide confinement. On the market, there are commercially available products offered by companies like Sika (Switzerland) or S&P reinforcement (Switzerland). They offer tapes (laminates) with width varying between 15mm-150mm, which are characterized by tensile strength around 2250-3000 MPa.

Before the product is applied, pull off tests of the concrete surface need to be conducted.



The result value of the pull of tests need to be >2 MPa. The reinforcement system also includes suitable adhesives accompanied by detailed instruction of application. The drawback of the actual available solutions is high price of reinforcement materials in comparison with traditional ones. However, in most cases, when the reduction of the cost due to ease of application and also advantages offered by very good durability of FRPs, is considered, it becomes a competitive solution to traditional methods.



Figure 16 Reinforced bridge using S&P carbon tapes

3.2 Application process

The behavior of a concrete structure strengthened with FRP systems highly depends on a proper preparation and profiling of the concrete surface. Flexural or shear strengthening of beams, slabs, columns, or walls, require an adhesive bond between the FRP system and the concrete. It needs to be highlighted that an improperly prepared surface can result in debonding or delamination of the FRP system before achieving the design load transfer. In order to apply FRP systems on concrete structures following steps need to be taken, see Table 2.

Table 2 Steps in application of FRP system

Step	Description
1	Repair of the corrosion. FRP laminates should not be applied to the structure that contains corroded reinforcing steel. The expansive forces associated with the corrosion process are difficult to determine and could compromise the structural integrity of the externally applied FRP system.
2	Injection of cracks. The cracks that are larger than 0,25 mm should be pressure injected with epoxy resin. If the structure is situated in aggressive environment smaller cracks may also require injection or sealing to prevent corrosion of existing steel reinforcement.
3	Surface preparation. Dust, dirt, oil, curing compound, existing coatings, cement layer and any other matter that could interfere with the bond of the FRP system to the concrete should be removed. Surface preparation can be accomplished using abrasive or water



	and sand-blasting techniques. If the FRP laminate is wrapped around the corners of rectangular cross section, the corners need to be rounded to prevent stress concentrations in the FRP system and voids between the FRP system and the concrete.
4	<i>Application of primer.</i> It should be applied to all areas on the concrete surface where the FRP system is to be placed. The primer should be placed uniformly on the prepared surface at the manufacturer’s specified rate of coverage. The applied primer should be protected from dust, moisture, and other contaminants prior to applying the FRP system. (a)
5	<i>Application of putty.</i> Bug holes and voids should be filled with epoxy putty; also roughened corners should be smoothed with putty. (b)
6	<i>Preparation and application of first layer of saturator/adhesive.</i> The saturator/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. The saturating resin should be applied uniformly or “roof-shaped” to all prepared surfaces where the system is to be placed. (c)
<i>Wet layup system/Pre-cured system</i>	
7	<i>Application of fiber sheet.</i> The fibers can also be impregnated in a separate process using a resin-impregnating machine before placement on the concrete surface or can be directly put on the surface with the use of roller. The Pre-cured system like: shells, strips, and open grid forms are typically installed with an adhesive. In next step entrapped air between layers should be released or rolled out before the resin sets. Sufficient saturating resin should be applied to achieve full saturation of the fibers. (d)
8	<i>Application of next layers of fiber sheet.</i> Second layer of saturating resin/adhesive is placed more or less after 30 minutes after application of first layer of fiber. The thickness of the second layer should be similar to the first one and varied circa 0.4-0.5 mm.
<i>Prepreg system</i>	
7	<i>Preparation and application of pre-impregnated fiber sheet.</i> Dry fiber sheet is impregnated in a separate process with the use of an impregnation machine. Prepregs can be stored in refrigerator at -20°C. Before application fiber sheet need to reach room



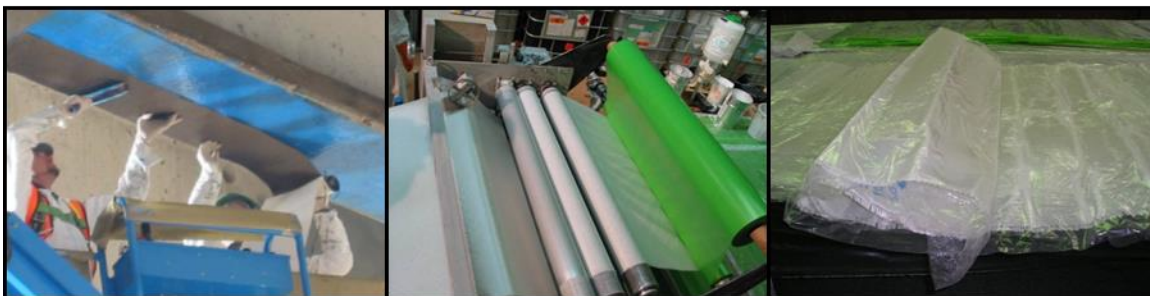
	temperature. (e)
8	Application of vacuum bag and curing. To remove the entrapped air and achieve uniform integrated material the vacuum bag can be installed and the pressure is applied. Prepreg systems should be cured at an elevated temperature. Usually a heat source is placed around the column for a predetermined temperature and time schedule. Temperatures are controlled to ensure consistent quality.
9	Application of optional topcoat. In order to enhance corrosion resistance external layer of topcoat can be applied.



1. Application of primer

2. Application of putty

3. Application of first layer of adhesive



4. Application of fiber sheet

5. Preparation and application of prepreg fiber sheet

3.3 Pre-stressed FRP systems

The concept of the pre-stressed FRP was introduced in 90s. High strength and good mechanical characteristic of FRPs combined with advantages offered by active strengthening based on pre-stressing, result in increased application of pre-stressed FRPs, although the necessity of anchorage systems can cause difficulties. There are a number of commercially available pre-stressed CFRP systems (for instance Sika, Figure 17) on the market. Application of pre-stressed laminates in order to increase bending capacity combine advantages of two technologies: passive reinforcement – bonded FRP laminates and active reinforcement – external pre-stressing. Due to pre-stressing effect, FRP laminates are able to participate in carrying existing sustained loads on the structure.



Anchorage system, transfers the pre-stressing force to the structure and thus it has to have reliable performance during the lifetime of the structure.

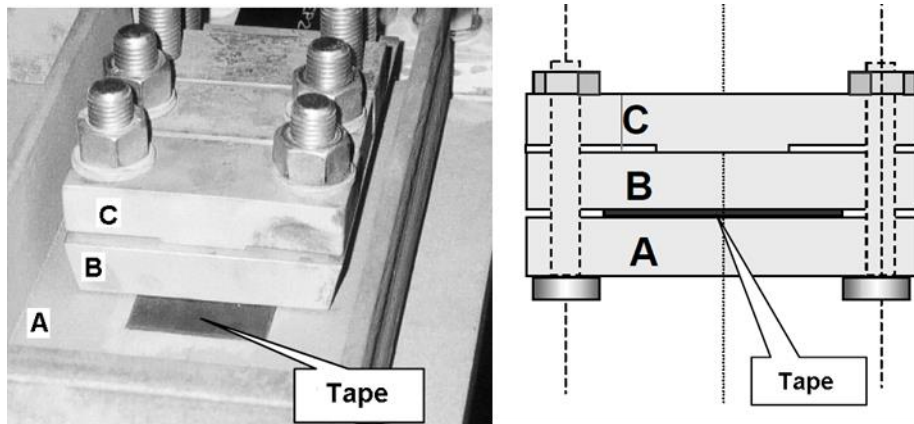


Figure 17 Anchorage detail in pre-stressed FRP systems (Sika)

The above presented systems can be based on three different types of fibers: carbon, glass and aramid. The choice of the material should be done by the engineer based on the structural condition and the service environment.

3.4 Challenges and Selection of FRP system

Environmental conditions uniquely affect resins and fibers of various FRP systems. The mechanical properties (for example, tensile strength, strain, and elastic modulus) of some FRP systems degrade under exposure to certain environments, such as alkalinity, salt water, chemicals, ultraviolet light, high temperatures, high humidity, and freeze and thaw cycles. The engineer should select an FRP system based on the known behavior of that system in the anticipated service conditions.

The performance of an FRP system over time in an alkaline or acidic environment depends on the matrix material and the reinforcing fiber. Dry, unsaturated bare or unprotected carbon fiber is resistant to both alkaline and acidic environments, while bare glass fiber can degrade over time in these environments. A properly applied resin matrix, however, should isolate and protect the fiber from the alkaline/acidic environment and retard deterioration. The FRP system selected should include a resin matrix resistant to alkaline and acidic environments. Sites with high alkalinity and high moisture or relative humidity favor the selection of carbon-fiber systems over glass-fiber systems.

FRP systems may have thermal expansion properties that are different from those of concrete. In addition, the thermal expansion properties of the fiber and polymer constituents of an FRP system can vary. Carbon fibers have a coefficient of thermal expansion near zero while glass fibers have a coefficient of thermal expansion similar to concrete. The polymers used in FRP strengthening systems typically have coefficients of thermal expansion roughly five times that of concrete.



Calculation of thermally induced strain differentials are complicated by variations in fiber orientation, fiber volume fraction (ratio of the volume of fibers to the volume of fibers and resins in an FRP), and thickness of adhesive layers. Thermal expansion differences do not affect bond for small ranges of temperature change, such as $\pm 30^{\circ}\text{C}$. Glass Fiber Reinforced Polymers (GFRP) and Aramid Fiber Reinforced Polymers (AFRP) are effective electrical insulators, while Carbon Fiber Reinforced Polymers (CFRP) is conductive. To avoid potential galvanic corrosion of steel elements, carbon based FRP materials should not come in direct contact with steel.

When it comes to the impact tolerance AFRP and GFRP systems demonstrate better tolerance to impact than CFRP systems. However CFRP systems are highly resistive to creep rupture under sustained loading and fatigue failure under cyclic loading. GFRP systems are more sensitive to both loading conditions [2].

Critical points in application of FRP systems are detailed preparation of the concrete surface. As described earlier in section 3.2 improperly prepared surface can contribute to debonding and not transferring the load to the FRP. Also due to the fact that the reinforcement is composed of fibers and resin, conditions of occupational safety and health need to be strictly obeyed. Depending on the construction type and fire requirements, FRP systems versus traditional techniques need to be studied. Protection of the FRP system in case of fire should be taken into account during design process. Application of FRP solutions is often associated with significant increase of (flexural) strength of the cross section, while the stiffness of the cross section is only slightly increased. This can cause significant deflections and has to be accounted in strengthening design.



4 References

- [1] Bavarian, B., Reiner, L., 2002, Corrosion Protection of Steel Rebar in Concrete using Migrating Corrosion Inhibitors, MCI 2021 & 2022, Cortec Corporation.
- [2] The Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures”, 2002, Reported by ACI Committee 440.
- [3] Radomski, W., 2002, Bridge Rehabilitation. Imperial College Press, ISBN 1860941222.