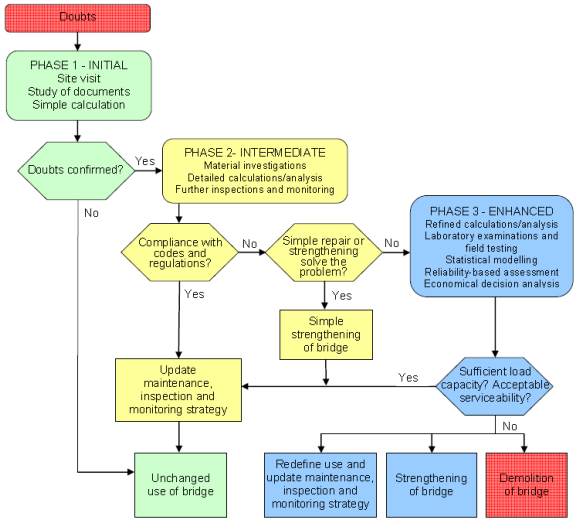
MAINLINE

MAINtenance, renewal and Improvement of rail transport iNfrastructure to reduce Economic and environmental impacts

Collaborative project (Small or medium-scale focused research project)

Theme SST.2011.5.2-6.: Cost-effective improvement of rail transport infrastructure



Assessment

Strengthening

Deliverable 1.4: Guideline for application of new technologies to extend life of elderly rail infrastructure

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Dissemination Level	
PU	Public

Abstract of the MAINLINE Project

Growth in demand for rail transportation across Europe is predicted to continue. Much of this growth will have to be accommodated on existing lines that contain old infrastructure. This demand will increase both the rate of deterioration of these elderly assets and the need for shorter line closures for maintenance or renewal interventions. The impact of these interventions must be minimized and will also need to take into account the need for lower economic and environmental impacts. New interventions will need to be developed along with additional tools to inform decision makers about the economic and environmental consequences of different intervention options being considered.

MAINLINE proposes to address all these issues through a series of linked work packages that will target at least €300m per year savings across Europe with a reduced environmental footprint in terms of embodied carbon and other environmental benefits. It will:

- Apply new technologies to extend the life of elderly infrastructure
- Improve degradation and structural models to develop more realistic life cycle cost and safety models
- Investigate new construction methods for the replacement of obsolete infrastructure
- Investigate monitoring techniques to complement or replace existing examination techniques
- Develop management tools to assess whole life environmental and economic impact.

The consortium includes leading railways, contractors, consultants and researchers from across Europe, including from both Eastern Europe and the emerging economies. Partners also bring experience on approaches used in other industry sectors which have relevance to the rail sector. Project benefits will come from keeping existing infrastructure in service through the application of technologies and interventions based on life cycle considerations. Although MAINLINE will focus on certain asset types, the management tools developed will be applicable across a broader asset base.

Partners in the MAINLINE Project

UIC, FR; Network Rail Infrastructure Limited, UK; COWI, DK; SKM, UK; University of Surrey, UK; TWI, UK; University of Minho, PT; Luleå tekniska universitet, SE; Deutsche Bahn, DE; MÁV Magyar Államvasutak Zrt, HU; Universitat Politècnica de Catalunya, ES; Graz University of Technology, AT; TCDD, TR; Damill AB, SE; COMSA EMTE, ES; Trafikverket, SE; Cerema (ex SETRA), FR; ARTTIC, FR; Skanska a.s., CZ.

WP1 in the MAINLINE project

The main objective for WP1is is to apply new technologies to extend the life of elderly rail infrastructure. This main objective can be subdivided in the following way:

- to explore and evaluate new technologies to extend the life length,
- to develop new more accurate assessment methods to determine if and when the life can be extended with and without any interventions (as e.g. strengthening),
- to further develop new technologies that can reduce life cycle costs for repair and strengthening and minimize the necessary traffic interruption,
- to develop a guideline for the application of new technologies to extend the life length,
- to transfer existing knowledge of new technologies to Eastern Europe and developing economies.

The pictures on the cover illustrates assessment and strengthening of structures

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Glossary

Abbreviation/ acronym	Description
ACI	American Concrete Institute
CEN	European Committee for Standardization
DoW	Description of Work
EC	European Commission
fib	International Federation for Structural Concrete
IABMAS	International Association for Bridge Maintenance and Safety
IABSE	International Association for Bridge and Structural Engineers
IALCCE	International Association for Life-Cycle Civil Engineering
IM	Infrastructure Manager
LCA	Life Cycle Analysis
LCC	Life Cycle Cost
LCCA	Life Cycle Cost Analysis
LCAT	Life Cycle Assessment Tool
RILEM	International union of laboratories and experts in construction materials, systems and structures
SIA	Swiss Society of Engineers and Architects
SB	Sustainable Bridges, EC FP6 Project
TecRec	Technical Recommendation approved as standard by UIC and UNIFE
UIC	International Union of Railways
UNIFE	Association of the European Rail Industry
WP	Work Package

1. Executive Summary

There are many traditional technologies available to extend the life of elderly rail infrastructure, some of which are being improved or developed, whilst new technologies continue to emerge.

In this guideline some of the most promising new or updated technologies are presented for bridges, track and earthwork regarding:

- Assessment methods
- Repair and Strengthening methods

In an Appendix strengthening methods are presented in more detail with examples of design calculations and work carried out.

The guideline is based on work presented in earlier reports in MAINLINE: ML-D1.1 (2013): *Benchmark of new technologies to extend the life of elderly rail infrastructure,* ML-D1.2 (2013): *Assessment methods for elderly rail infrastructure* and ML-D1.3 (2014): *New technologies to extend the life of elderly infrastructure*; In these reports, background information and more references can also be found.

2. Acknowledgements

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3. Introduction

3.1 General

The MAINLINE project has the overall aim to give railway infrastructure owners tools to reduce environmental footprint and prolong the life length of existing rail infrastructure

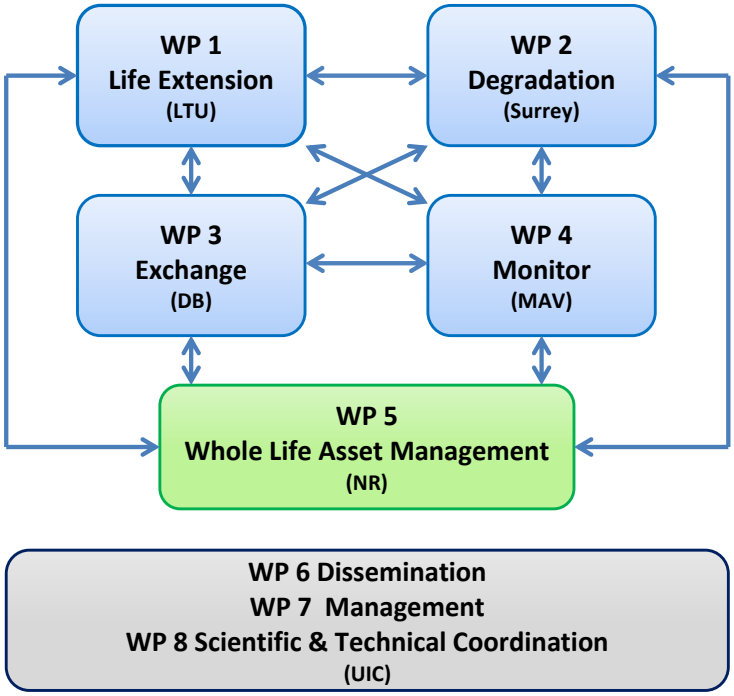


Figure 3-1. General organization of the project

The purpose of this report is to give a guideline to technologies that have the best potential of achieving this aim regarding methods to prolong the life of existing elderly rail infrastructure. A chart illustrating the project is given in *Figure 3-1.*

WP1 has interacted with the other WPs helping to identify new technologies, methods and data that have the potential to improve degradation and structural models (WP2), replacement (WP3), monitoring (WP4), whole life environmental and economic asset management (WP5) and dissemination of results (WP6). Results are uploaded on the web, see http://www.mainline-project.eu/Results,7.html.

The partners in WP1 are listed in Table 3-1.

Table 3-1. Partners in WP1

Part n°	WP1 Partners	Country
1	Union Internationale des Chemins de Fer - UIC	France
2	Network Rail Infrastructure LTD - NR	United Kingdom
7	Universade do Minho - UMinho	Portugal
8	Luleå Tekniska Universitet - LTU	Sweden
11	Universitat Politecnica de Catalunya - UPC	Spain
19	Skanska AS - Skanska	Czech Republic
20	Jacobs/SKM	United Kingdom

3.2 Outline

In this Guideline, new technologies are presented that have become available during recent years to extend the life of existing elderly rail infrastructure. The report is summary and a follow up of earlier reports: ML-D1.1 (2013) "*Benchmark of new technologies to extend the life of elderly rail infrastructure*", ML-D1.2 (2013) "*Assessment methods for elderly rail infrastructure*" and ML-D1.3 (2014) "*New technologies to extend the life of elderly rail infrastructure*"

Some of the technologies have earlier been introduced in the EC-FP6 projects Sustainable Bridges (2007) and Innotrack (2010). Others have been reported in a questionnaire to railway infrastructure owners; see Appendix A. in ML-D1.1 (2013). Still others are treated in three parallel EC projects: Automain (2012), Smartrail (2012) and Sustrail (2012). Of these, Automain and Sustrail concentrate on track, while Smartrail has a scope more similar to that of MAINLINE. Smartrail focus on earthwork, transition zones, slopes and tunnels and MAINLINE focus more on bridges and track. Both projects work with life cycle cost assessment, LCCA.

Assessment methods for bridges are first treated in chapter 4. Strengthening of structures is then treated in chapter 5. Tunnels are treated in chapter 6 and track is treated in chapter 7. In Appendix A Finally references are given in Appendix B.

Masonry arch bridges are not treated here but much information can be found in a new guideline *Recommendations for the inspection, assessment and maintenance of masonry arch bridges,* UIC 718-3 (2014),

An example of how operation and maintenance of bridges can be organized is given in Figure 3-2 and Figure 3-3.

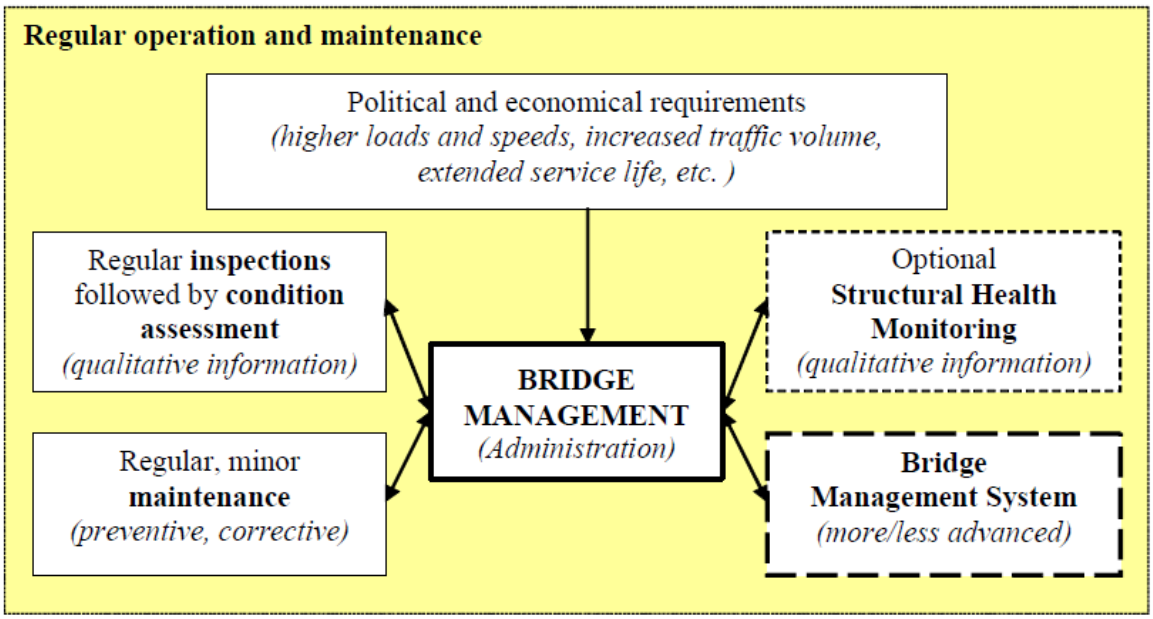


Figure 3-2. Regular operation and maintenance of bridges. If there are questions regarding serviceability, action can be taken according to Figure 3-2. UIC 718-4 (2009).

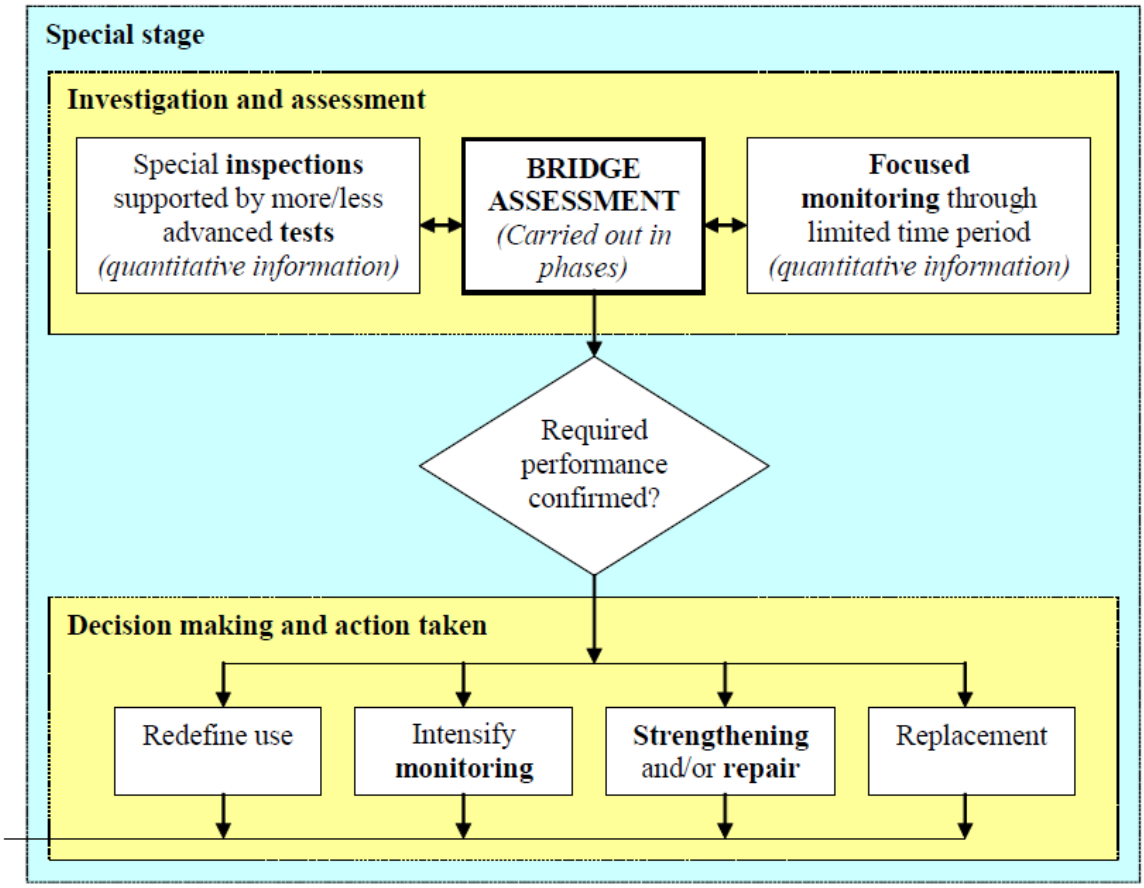


Figure 3-3. Special stage of operation and maintenance of bridges when there is a special concern regarding, safety, serviceability or durability. After decisions are made and actions taken (the last line in the figure), the bridge is returned to regular operation and maintenance according to Figure 3-2. UIC 718-4 (2009).

4. Assessment of Structures

4.1 General

The assessment of existing infrastructures is becoming essential as their stock is increasing day after day. The assessment process can be more or less sophisticated, cumbersome and accurate, depending on the asset to be evaluated and the required information to be obtained. In the context of the MAINLINE project, where the optimum management strategies from an economic point of view are foreseen, the costs incurred in the assessment and the information obtained take maximum relevance. Therefore, different levels of assessment should be considered depending on the chosen results and the consequent cost. In this Guideline, the advanced assessment methods are also considered, see Figure 4-1.

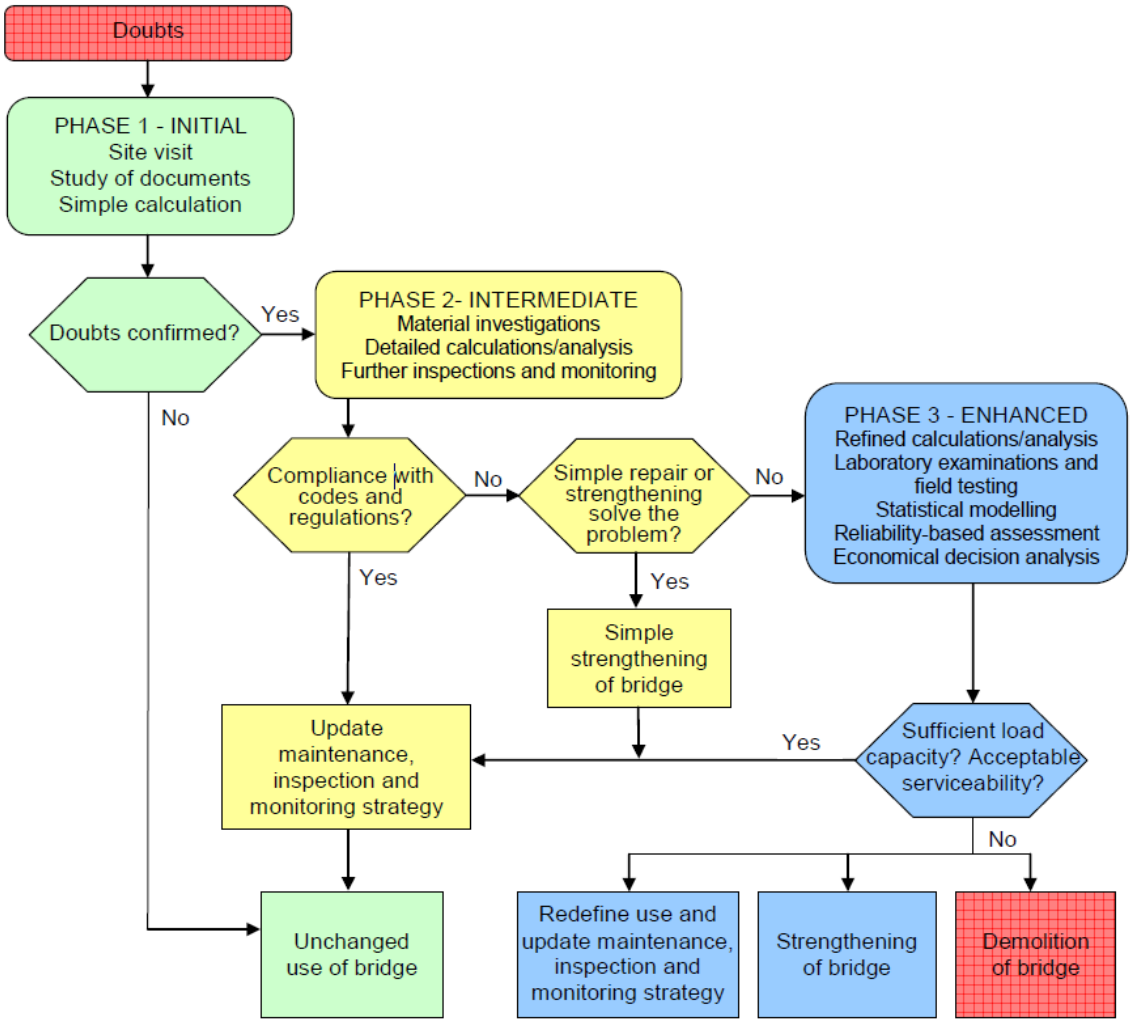


Figure 4-1. Flow chart for the assessment of existing structures. Three phases are identified: Initial, Intermediate and Enhanced, depending on the complexity of the questions involved. SB-LRA (2007), UIC 778-4 (2009).

Assessing the safety of an existing bridge for determining its load carrying capacity, evaluating its ability to support increased loading, or prolonging its service life, should evolve to become an adaptive, multi-level process that allows for the refinement of an engineer's initial estimate of the present and future state of the bridge and its behaviour. This is, for instance, the procedure adopted in the Sustainable Bridges project (SB-LRA 2008).

At the Initial Level, the assessment is usually performed using standard methods similar to those used in design. If the bridge passes the Initial Level assessment, no additional analyses or actions are necessary and the bridge remains in operation as it is. Bridges that fail to pass initial safety checks should be re-evaluated using advanced assessment methods, which would involve any combination of the following methods (Wisniewski et al. 2012):

- Additional more thorough inspections with possible field testing for material properties to obtain better estimates of member strengths.
- The use of Weigh-In-Motion (WIM) or other monitoring techniques data (for instance, temperature) to obtain improved estimates of the applied loads.
- The use of refined structural analysis models or field measurements to obtain better estimates of the response of the bridge to the applied loads.
- The use of reliability techniques to estimate the probability of exceeding an specific limit state

It is evident that increasing the level of assessment will require more resources for advanced experimental methods, theoretical analyses. The decision of going further to the next level should be supported on the saving of resources derived from the final decision. An equilibrium is foreseen between the new expenses derived from the more advanced assessment and the savings because the final decision may result on a lower strengthening or repair need. Therefore, in the framework of a Life Cycle Assessment (**LCA)** as the one proposed in the MAINLINE project, the decision on the use or not of a more advanced assessment method will be based upon the following criteria:

1.- To minimize the total cost, expressed as:

$$C_{total} = C_{cons} + C_{insp} + C_{ass} + C_{user} + C_{repair} + C_{failure} \quad (4.1)$$

C_{cons}= construction cost

C_{insp}= cost of inspection and routine maintenance

C_{ass} = cost of assessment

C_{user} = user cost

C_{repair} = cost of repair, strengthening,…

$C_{failure}$= cost associated to the failure of the bridge to perform a required limit state

2.- with the following constraints: To guarantee a minimum performance level (safety, service) to the user

$$S > S_0 \quad (4.2)$$

In the presented framework, it makes then sense to look to the feasibility of enhanced or advanced assessment as a mean of reducing the repair and failure costs, and consequently, the total life-cycle cost. Therefore, advanced assessment may be seen as the most effective and cheapest way to extend the service life of an existing bridge, before anything else is done, allowing, in some cases, the continuation of the bridge in its normal service operation, without additional expenses. This justifies the inclusion of advanced assessments in the LCA of existing railway bridges.

The assessment of bridges, like other structures, is still usually done using approaches which were originally developed for design. The analytical procedures used in design are often conservative. This approach does not suppose a large increment of cost since placing more material (steel, concrete,…) during the construction of a new bridge is not too much expensive compared to labour cost. However, this is not the case when retrofitting structures already in service. Alternative approaches are available which give more realistic results. The high cost of strengthening existing structures makes it more likely to be worth using them in assessment.

For some advanced assessment methods, only direct economic costs and environmental impact costs will be considered (no user costs). This will be the case when the assessment does not require to stop the normal traffic operation in the bridge. However, some advanced assessment techniques requiring the intervention in the bridge will derive in possible user costs to be considered. The environmental impact costs (benefits) come from the fact that knowledge of actual capacity of the asset may derive in large economic savings coming from no need of repair/strengthening/demolition.

The aim of this Deliverable ML-D1.4 is to provide a catalogue concerning maintenance strategies to extend the service life of predefined existing types of assets. Proper maintenance of existing structures ensures that the service life of the assets may be extended considerably. Such maintenance should be carried out with due attention to the total cost and environmental impact. Among the maintenance strategies, the one of “doing nothing” can be considered as the most economical effective and environmental friendly. Such strategy can be the result of an advanced assessment which reveals the asset still in good condition to continue its function without any up-grading.

According to Deliverable ML-D1.2, new methodologies for a more accurate assessment would involve one or a combination of the following tools and techniques:

- Direct application of reliability-based assessment methods
- Consideration of system safety, redudancy and robustness criteria
- Site-specific loads and impact factors
- Material and diagnostic load testing, including Structural Health Monitoring (SHM) and model updating
- Proof load testing

In the following a short description of these new technologies is presented. Further information is available in Deliverable ML-D1.2

4.2 Direct application of reliability-based assessment methods

The application of these methods is mandatory when the standard deterministic or semi-probabilistic methods declare the bridge as not satisfying relevant limit states and before any remedial repair or strengthening is undertaken. With the application of this method, the reliability index, related to the bridge safety, is used as the performance index and a minimum target value is indicated as providing the minimum safety required for the structure. Therefore, with the application of this method, the performance index is directly obtained and no subsequent analyses are needed.

A complete description about how probability-based assessment may be carried out can be found in the deliverable SB-LRA (2008) from SUSTAINABLE BRIDGES and the deliverable from SMARTRAIL Project on “Development of a General Rail Transport Infrastructure Safety Framework”. In this last case, probability-based assessment is applied to railway bridges and embankments. A guideline showing how this method is applied on existing bridges can be found in deliverables D1.2 and D1.3 of MAINLINE.

4.3 Consideration of system safety, redundancy and robustness criteria

The importance of considering structural robustness and redundancy in the assessment processes is highlighted by a number of historical events that did not derive in catastrophic collapse following local failures in critical members. As the standard assessment methods are based of a member failure criteria, the new proposed technology uses the measure of system safety and robustness as a modifying factor in the rating equation.

Significant discussion exists on the advantages and disadvantages of different methodologies to evaluate robustness. Three different levels of analysis can be used to quantify robustness: risk analysis, probabilistic analysis and deterministic analysis (see deliverable ML-D1.2).

Current research is focusing on developing methods for assessing the robustness of bridge systems and improving the bridge design and safety evaluation process by accounting for their structural robustness properties. This focus is justified because structural robustness is a property of the structural system that is appropriate for different types of hazards. The goal of current efforts is to provide engineers with the necessary tools for the consideration of bridge system safety and structural robustness in bridge engineering practice. Because probabilistic methods cannot be used for the practical assessment of bridge safety on a regular basis, researchers and code writers have developed deterministic methods of analysis which are calibrated to lead to similar conclusions concerning a bridge’s safety. In either case, whether probabilistic or deterministic methods are being used, a necessary step for considering structural robustness is to define appropriate non-subjective measures of robustness and develop acceptance criteria (see ML-D1,2 and ML-D1.3). A complete description is presented in Anitori et al. 2013.

In Cavaco et al. (2013 a,b), it is explained how most of the proposed measures of robustness are relative, in the sense that they may help identify which structure is more or less robust than another. However, a target or threshold value that defines the border between what is

robust or not, does not exist. However, when dealing with life-cycle assessment and looking at the best maintenance/repair option for a group of assets, this is not a problem, since optimal decisions can be taken just on a relative basis.

Simplified deterministic calibrated methods should be provided to allow engineers who are not trained in probabilistic methods, to perform direct evaluations using widely available tools. Alternatively, the code could help engineers by providing guidelines that will help engineers determine the level of robustness without the need to perform special analyses. Thus, the alternative methods can be divided into: a) application of system factors to design and evaluation equations; b) Direct deterministic analysis method and c) probabilistic analysis methods.

System factors: This is the easiest and least time-consuming approach, it allows the practicing engineer to evaluate the robustness of the structure by means of characterizing it among a set of standard topologies covered by the code for bridges with well-known behavior. The prescriptions of detailing, local resistance and other influencing parameters should follow the identification of the structure in the code. The final output is a system factor that account for inherent robustness. The system factor is then used in the assessment equation at member level. Example of this procedure can be found in AAHSTO 2003 and Casas et al. (2012).

Deterministic direct analysis method: When the structural topology is not covered by the code or a higher level of refinement is required, a numerical analysis should be carried out. Although this analysis should be as simple and fast as possible it must include post elastic behavior of the materials and be able to describe properly the overall response of the original and damaged structure. An example on how the robustness can be evaluated is presented in appendices B and C of deliverable D1.3. An example of application of the inherent robustness to the advanced assessment of a railway bridge, using a deterministic approach, can be found in Wisniewski et al. (2006). The same example with a simplified probabilistic approach is presented in Wisniewski et al. (2009). Other simplified methods to consider the system behavior and redundancy of existing bridges in an advanced assessment are also presented in SB-LRA (2008), SB4.4.1 (2007), Wisniewski et al. (2009) and Casas and Wisniewski (2013).

Reliability based method: Reliability criteria should be provided by the code as the basis for performing the analysis. The level of complexity is the highest, including the simulation by numerical methods and the consideration of uncertainties in the structural parameters. Examples can be found in SB-LRA (2008).

At the present moment, only 2 bridge Codes (Canada and USA) provide practical and clear indications on how to take into account the redundancy effects into the assessment process. In the first case, it is done by the definition of a specific target reliability index; in the second, the concept of system factor is used. This second approach seems the most effective for a practical use on an advanced assessment process. This was the approach also considered in the SUSTAINABLE BRIDGES project. In the deliverable SB-LRA (2008) are presented the guidelines and the method to follow for the consideration of system performance and redundancy to a specific bridge assessment, based on the concept of the redundancy factor (Deterministic direct analysis method). Practical applications are presented in Wisniewski et al. 2006, Wisniewski et al. 2009, showing how taking into account their redundancy and robustness characteristics, bridges that would be condemned to a strengthening or

replacement could still be assessed as safe and therefore kept in service without further actions.

Improving the performance of a bridge system that shows low levels of structural reliability, or can be achieved by one of three ways:

1) Reducing its exposure to the relevant hazard. For example this can be achieved by placing barriers around critical bridge members or columns to protect them from potential impacts or reducing the access for potential malicious activities.
2) Reducing the vulnerability of the bridge members to particular hazards. For example, this can be achieved by wrapping members with steel or FRP jackets to reduce their vulnerability to direct impact and increasing their ductility to improve their capability of withstanding seismic motions.
3) Enhancing the robustness of the system. This can be achieved by adding members to change the configuration of the system and ensure the presence of alternate load paths. Other approaches to improving the robustness may consist of providing adequate mechanisms for load transfer through improving the ductility of bridge members and providing adequate detailing and connections.

4.4 Site-specific live loads (WIM), dynamic amplification factors and temperature effects

The loading to which bridges are subject is known to be an area in which significant savings may be made due to necessary conservatism of bridge loading standards that are broadly applied. Using measuring data, the load effect, or effects, on a particular bridge, or a range of bridges, can be more accurately estimated. Knowledge of the current loadings to which railway bridges are subjected is imperative for accurate bridge evaluation. The live loads and dynamic amplification factors in the design codes are given for the design of new structures and can therefore be very conservative in some circumstances leading to structures failing their assessments. Consequently, it is often beneficial to use site-specific live loads and dynamic amplification factors when assessing existing railway bridges.

To collect information on the site-specific live loads, Weigh-in-Motion (WIM) systems are commonly used. WIM is the process of converting an instrumented track or bridge into a scale for weighing passing trains. Such systems provide information on:

- Static and dynamic axle loads (or bogie loads depending on type of system)
- Axle distances (or bogie distances depending on type of system)
- The speed of the train
- The direction of the train

In chapter 4 of the Sustainable Bridges background document SB4.3.2, (2007) and Gonzalez (2011), a method is presented by which one is able to determine site specific characteristic train loads from BWIM measurements.

In many cases, the assessment of bridges at Ultimate Limit States (ULS) is of interest. To this end, an accurate estimate of the maximum traffic effect within a predefined period of time is necessary. Based on the site-specific train loads obtained using WIM techniques, a simple but enough accurate method is presented in Deliverable ML-D1.2 to derive such maximum

effects. In the case of railway infrastructure, because the probability of meeting trains in bridges with more than 1 track is very low, when the volume of train runs is high, the one-train load governs.

In the deliverable from SMARTRAIL Project on “Development of a General Rail Transport Infrastructure Safety Framework” (SmartRail 2013) are presented other topics related to railway loading and alternative methods to use of WIM data to define maximum loading scenarios.

The dynamic amplification factors (DAF) prescribed in design/assessment codes are sometimes based on theoretical analysis and/or dynamic load tests of existing bridges and tend to be conservative. There is considerable discrepancy among the values recommended by different codes due to the complexity of the Vehicle Bridge Interaction (VBI) problem. They typically suggest a dynamic amplification which is function only of a few general parameters that ignore many of these significant bridge and truck dynamic characteristics. Thus, the DAF values are conservative and they produce maximum dynamic effects that might not necessarily correspond to the maximum static effects. This level of conservatism could be acceptable for new construction but not for the assessment of existing ones

By measuring displacements or strains, site-specific dynamic amplification factors (DAF) can be calculated. This can be done either from (a) tests with trains crossing at normal running speeds and tests with the same trains crossing at a crawling speed (typically less that 10 km/h) or (b) by filtering the measured signal to separate the static from the dynamic content. The latter method is more delicate and requires more experience in signal analysis. Moreover, a total separation of the dynamic response can be hard to achieve by filtering on some bridges. Consequently, this method is only recommended when it is observed that the dynamic response constitutes oscillations around the static response. The DAF is then calculated by dividing the maximum dynamic response with the maximum static response.

The dynamic behaviour of the bridge under traffic loads consists in absorption, storage, dissipation and release of energy that is stored in the structure due to dynamic traffic action. For elastic bridge behaviour, this energy stored in the bridge element consists in vibrations leading to increase of deflections and internal forces. Elastic bridge behaviour is considered for fatigue and service limit state. However, for the ultimate limit state, elastic-plastic structural behaviour must be accounted for, and formulas like those commonly given in design codes are then fundamentally wrong since they refer to elastic structural behaviour only (Brühwiler & Herwig, 2008).

Besides the correct assessment of variable-in-time loads as the live load or traffic load, in some cases, the accurate assessment of temperature effects is of interest, mainly for statically non-determined bridges. This is particularly relevant in the case of concrete bridges and for the bridge assessment versus the serviceability limit states (cracking, deflection, vibration). In fact, the cracking due to temperature effects may produce a reduction on the sectional stiffness leading to possible adverse effects. The temperature effect is less relevant in the case of assessment versus ultimate limit states, except in the case of fatigue assessment. This is due to the relaxation of thermal stresses when releasing some of the imposed boundary conditions or decreasing the stiffness that occur when increasing the external load level. At the limit, when the bridge is close to failure, it becomes a mechanism, and therefore, the imposed strains due to temperature effects, do not produce internal forces in the bridge elements.

When designing a bridge for thermal effects, the temperature distribution has historically been considered in similar ways in Europe (CEN 1996). The distribution can be divided into parts: an average temperature, horizontal and vertical temperature gradients/differentials, and a non-linear residual temperature gradient/differential.

The average temperature component, which governs the longitudinal movements, has been coupled to the ambient air temperature in most national codes (CEN 1996). In the design process the average temperature is determined depending on the geographical location of the bridge. The main differences between the various national building codes that preceded the Eurocode lie in the application of the temperature differentials. The differences in design approach between countries in Europe could be a considerable factor for the assessment of existing bridges, since the use of various differentials may give differences in the structural response.

For assessing the temperature influence of existing railway bridges the most common methods is the finite difference method (Emerson 1973) and finite element modelling. In most studies the finite element method has been used for both calculating the temperature distribution and the resulting stress field (Elbadry and Ghali 1983, Sveinson 2004, Larsson 2012).

The restraint causing the large thermal stresses is significantly reduced if cracks are present in the structure (Jokela 1983). The cracks reduce the moment of inertia and thus the stiffness, leading to a reduction of the stresses. This must be considered when using a linear-elastic model for evaluating the effects of thermal actions, since these effects otherwise may be overestimated.

Other important studies of longitudinal effects in concrete bridges have been performed by Prakash Rao (1986) and Thurston et al (1984). Prakash Rao found that differences in temperature between members (walls and slabs) in a bridge could cause severe effects and lead to cracking. It was also found that it is important to consider both positive and negative temperature gradients when assessing temperature effects. Thurston et al. developed a method for calculating thermal stresses in cracked beams and bridges. The background to the development was the intention to use partially prestressed concrete bridges, to allow controlled cracking to take place. The results found in the investigation showed that cracks in critical sections reduced the thermal moments with up to 46 % for a large temperature load consisting of a gradient of 40 °C. This confirmed that cracks in a concrete bridge can reduce the induced restraint and thus reduce the effects of thermal actions. Casas (1983) demonstrated that the temperature effects in the case of prestressed (full prestressed and partially prestressed) concrete bridges built by the balanced cantilever method were almost negligible when carrying out a non-linear analysis up to failure.

An attempt to determine which temperature and climate situations who give the most unfavorable situations was presented in Larsson (2012). Input data and results from long-term temperature simulations and corresponding thermal stress simulations were analyzed to find which combinations of climate factors that gave the most unfavorable stresses. It was found that a large influx of solar radiation combined with clear skies and a large difference in air temperature gave large thermal stresses, which was expected. However, another situation was also found, where a rapid increase in temperature during winter time also produced large stresses. This situation could be the cause of cracks due to thermal effects in locations where the effects of solar radiation are limited. The results in Larsson (2012) also showed that instead of using fixed values of air temperature and/or temperature differentials

it is possible to use a shorter period of climate data to simulate thermal effects. A short-time period gave similar results to the long-term simulations, and may give more reasonable results than using fixed values in every situation. This fact could be of paramount importance in the assessment of existing bridges.

The effects of thermal actions in steel and composite bridges have not achieved as much focus as thermal effects in concrete bridges. The main reason for this is the more direct effects that can be seen in concrete structures from thermal actions, such as cracks on one side of the bridge only due to solar radiation affects, with also the consequences of thermal effects being less severe for steel bridges.

The most common assessment method concerning thermal effects in steel and composite bridges is the finite element method. The same principles concerning the average temperature and temperature differential as for concrete bridges is applicable for steel and composite bridges, with slightly altered values depending on the difference in material properties. It should how ever be noted that investigations exists where the design values given in e.g. Eurocode is deemed to be underestimated (Lucas et al. 2003)

For a steel bridge the main factor is the longitudinal elongation and retraction. Due to a much higher heat conductivity in steel than in concrete, the effects of changing temperature is more direct and the effects arise more quickly. If the bridge is restrained from moving in the longitudinal direction, large stresses and forces will occur that may lead to risks for failure is the steel components and/or in other parts of the structure. (CEN 1996)

A major concern for steel structures is the damage from fatigue. Studies of fatigue due to temperature variations in integral abutment steel bridges have been performed during the latest years by Hällmark et al (2010), Dicleli and Albhaisi (2004). Here it was shown that low cycle fatigue from temperature variations could significantly reduce the lifetime of such a bridge. However, it was also shown in the study by Hällmark et al. that the risk for failure due to low-cycle fatigue is quite low for bridges with a length below 50 meters. Large effects on the amount of fatigue were also found dependent on the surrounding soil stiffness.

4.5 Advanced models (FEM). Model updating and incorporation of data from inspection and monitoring

Up-dating of models for resistance, loading and structural response to get more accurate theoretical models in the analysis is also considered as a step further in an advanced assessment.

Model updating can be also carried out via diagnostic load testing. This type of test provides useful information when structural models including finite element methods can not accurately predict the behaviour due to uncertainties in member properties, boundary conditions and influence of secondary members. As an example, in Olaszek et al. (2013) are presented the results of 3 diagnostic load tests in different bridge structures, emphasizing their diagnostic potential for assessment. In the cases presented, it is shown how the experimental results differ considerably from the expected ones and as a result, up-dated models are obtained based on the results of the tests.

Two emerging measuring technologies are of main interest in concrete bridges: acoustic emission and distributed optical fiber. Recent experimental studies have shown how intensity analysis of hits of acoustic emission can detect and quantify the current condition of the bridge as well as taking into account the degree of existing damage prior to conducting a load test. Also the b-value analysis can provide an early warning of damage accumulation. Finally, AE source triangulation enables the location and pattern of cracks (ElBatanouny et al. 2014). Another technique that is able to detect cracking before being visible, and to locate the cracks and their width is the distributed optical fiber system OBR (Optical Bakscattered Reflectometer). The technique has the feasibility of measuring strain and temperature continuously along the fiber, therefore detecting the crack when it appears in the concrete element. The bonding of a continuous fiber along the element allows to detect the crack wherever it appears (Villalba and Casas 2013, Rodríguez et al. 2014). The technique of Smart film with crisscross enameled copper wires glued on the surface of a concrete structure can also monitor the initiation time, length propagation, shape and location of cracks (Zhang et al. 2014).

One challenge in dealing with SHM has been the processing of the large amount of data extracted and interpreting it into meaningful information that can be used for decision making. Accordingly, SHM has enjoyed significant research efforts aimed at the development of algorithms and methodologies for system identification, damage detection, and updating of finite element models. One of the main problems related to the use of SHM data in the damage detection is to find out a real-time strategy to conduct structural assessment without the need to define a baseline period in which the monitored structure is assumed healthy and unchanged. This can be achieved by means of machine-learning algorithms know as cluster analysis. They are able to find groups in data relying only in its intrinsic features (Sohn and Kim 2008). A change in the structural response, due to a possible damage in the structure, is reflected in these intrinsic features and derives on the aggrupation of data from the SHM in clusters different from the previous ones. This derives in an on-line and real-time damage detection.

Only recently, research that treats SHM under uncertainty has emerged. However, these studies have mainly focused on information related to the load effects that SHM provides. Okasha and Frangopol (2012) have proposed an approach in which the SHM information can be, in fact, used to update the structural parameters of the structure that are in turn used in updating the lifetime reliability of the structure. The information provided by monitoring a bridge can be used for updating the PDF (Probability distribution function) of its time to failure through a Bayesian process.

A well-known way to incorporate SHM data into a life-cycle assessment is by means of Bayesian updating techniques. However, judgment based only on SHM data obtained over a period of time may lack information on events that are encountered outside this period and, therefore, it is crucial to combine SHM with prior estimates of these quantities. The classical estimation approach treats the parameters of the PDF (probability distribution function) deterministically and makes not possible to incorporate prior information of the uncertain variable. Instead, the Bayesian estimation approach treats the parameters as random variables and, in this way, makes it possible to user prior knowledge.

A Bayesian updating framework (SB-LRA, 2008) can be used to consider both the original and posterior sources of uncertainty in a consistent manner, resulting in a more reliable indication of the actual properties of the materials, improved estimation of the resistance of the bridge members, and a more accurate understanding of the stress and load distribution throughout the structure.

The treatment of SHM data is usually associated with monitoring of extreme events (i.e. load effects of very heavy trucks). The literature of papers linking the themes of Bayesian updating and extreme value modelling is sparse, in part due to computational difficulties, some of which have recently been overcome by techniques such as Markov chain Monte Carlo (McMC) (Bocchini, et al., 2013). Extreme value distributions do not lend themselves easily to Bayesian updating; the main problem is that there is no conjugate distribution. For example, the Weibull distribution or the Gumbel distribution are the most used extreme values distributions for model the traffic extreme events. The Bayesian-updating of extreme value distributions cannot lead to explicit posterior distributions. Hence, a simulation procedure is the best way to determine the posterior distribution. The Metropolis- Hastings (MH) algorithm has been suggested for this purpose.

4.5.1 Advanced FEM models

An accurate and advanced assessment requires the most accurate response models to be used. The finite element method (FEM) has become widely used for design and assessment of bridges. Finite element (FE) analyses provide the possibility for more accurate studies of the structures than what is possible with more traditional methods used for structural assessment. In such an analysis, the three-dimensional geometry of the structure and its non-linear response due to e.g. material plasticity, cracking and second order effects can be taken into account.

Assessment of structural safety and functionality of existing bridges is a step-level procedure. FE analysis can be helpful at intermediate level assessment for structural (system) analysis, in combination with resistance models. In bridge design, 3D linear FE analysis corresponding to this level is commonly used. However, FE analysis is particularly useful for advanced assessment. Here, non-linear analysis is usually needed. This requires skilled and experienced structural engineers and is considerably more time consuming. On the other hand, a non-linear (FE) analysis is considered to have the highest potential for discovering any additional sources for load carrying capacity in reinforced concrete railway bridges (SB-LRA 2008).

The use of FEM for analysis of building and civil engineering structures is treated in several text- and handbooks, e.g. Crisfield (1991), Rugarli (2010) and Blaauwendraad (2010). Design and analysis of concrete structures are treated in e.g. Fib (2008) and Rombach (2004). In Sustainable Bridges (2007), assessment of railway concrete bridges with non-linear FE analysis is treated. Guidelines for non-linear finite element analysis of concrete structures are provided for girder members in Rijkswaterstaat (2012) and for modelling of shear and torsion in bridges in Broo (2008).

Structural idealisation

The most important step when assessing an existing structure using FEM is the definition of the structural model and its properties. The purpose of the structural analysis is to model the behaviour of the structure to a sufficient level of accuracy. Consequently, it is desirable and necessary to make simplifications in the structural model. The geometry, material and loading, as well as the extent of the structure and its boundaries may need to be idealised. Different structural models reflect different aspects of the structural behaviour differently well, and sometimes several models with different levels of detailing are needed. Often, structural models for assessment need to be different from models used in design.

The purpose with the analysis decides how the structural model should be defined and what idealisations that can be made. There are many aspects to consider when setting up a structural model, such as:

- What should the model be capable of describing?
- What structural phenomena should be reflected? (Is it static or dynamic? Does it involve large deformations, non-linear or time dependent material response, etc.?)
- Can the problem be simplified by using beam or plate theory, or a two-dimensional stress state?
- What is the extent of the model and how should the boundaries be defined?
- How should the material response be idealised and what material parameters should be used?
- How should interaction between different parts or materials be modelled?
- How can important details, such as supports, connections and stiffened areas be simplified?
- How should the actions on the structure be applied?
- What element types and FE mesh density is needed?
- What solution method should be used?

The verification of the FE model and analysis results is a very important step. Since advanced FE analyses are complex with many possible error sources, a rigorous quality control is needed. An FE analysis usually provides large amounts of results. Consequently, the results need to be post-processed and interpreted bearing the structural idealisations made in mind. It is of great importance that the structural engineer has a thorough understanding of the structural response in the model as well as the behaviour of the real structure.

Linear FE analysis

Guidelines for linear FE analysis of concrete structures can be found in Fib (2008), Rombach (2004), and for plate structures in Blaauwendraad (2010). The question of how to redistribute sectional forces and moments for design of reinforcement in concrete slabs, and how this is connected to the structural idealization and FE modelling is treated in Pacoste *et al.* (2012).

In reality, most bridge structures have a non-linear response under loading up to failure; concrete bridges normally have a pronounced non-linear response already for service loads due to cracking. In ultimate limit states, the use of linear analysis can normally be justified since the structures have good plastic deformability.

In the assessment situation, the structure is already designed. The original bridge design was most likely <u>not</u> based on detailed 3D linear analysis, but rather made with simplified analysis methods typical for the time of the design. This means that the bridge will most likely have larger capacities in some parts or sections and smaller in others, compared to the design based on linear elasticity. Both designs may fulfil equilibrium, but can have different requirement on the plastic deformation capacity; due to the non-linear structural response, it is not necessary that the linear analysis require smaller plastic deformations.

This means that there is a big risk of underestimating the capacity of the bridge when using linear FE analysis for assessment in the same way as it is used for design. The critical section, limiting the capacity according to the linear analysis, may not be as critical in reality

due to the non-linear response. If linear FE analysis is used for assessment, redistributions from the linear force distributions must be allowed, and conservative estimations of redistribution widths according to e.g. Pacoste *et al.* (2012) should be avoided. Comparison to other structural models must be included and knowledge of the structural model used in original design is very desirable.

Non-linear FE analysis

In a non-linear FE analysis the structural response is simulated in a more realistic way, with the possibility to take the material non-linearity into account e.g. due to steel yielding and concrete cracking or non-linear geometric effects. The stress redistribution in statically indeterminate structures is reflected, providing a more correct distribution of the load effects. Moreover, since a realistic material response is included, the resistance of the structure is determined by failure occurring in the overall structural analysis. This way, intermediate results in terms of cross-sectional forces and moments are not needed, avoiding the inconsistency of combining linear system analysis for determination of action effects with non-linear local analysis for determination of resistance. Non-linear FE analysis can be used for all types of bridges and for service limit state SLS as well as for ultimate limit state ULS.

In the Eurocodes, EN 1992-1-1 (2004), non-linear analysis is specified as the most advanced level of structural analysis, and general guidelines are given for such analysis. In research, non-linear FE analysis is long since regularly used to obtain a better understanding of structural behaviour. However, non-linear analysis is still not often used in engineering practice. It is more time consuming than simplified analysis methods and the demands on expertise of the structural engineers performing the analyses are higher. On the other hand, in many cases it has shown great potential in revealing additional load carrying capacity and to provide great cost savings in assessment of existing bridges, Plos (2002), SB-LRA (2008) and Broo et al. (2009).

Non-linear FE models can be made with different levels of detailing, leading to analyses reflecting the structural response and resistance to different levels of accuracy:

- For structural analysis of entire bridges, models built up of “structural” finite elements like beam and shell elements are often useful. Models on this level of detailing are well established and verified to reflect the response due to bending and normal forces in a good way; they are commonly used by researchers and engineers and the results are regarded as reliable. However, failure due to e.g. shear, anchorage or local buckling must be checked with separate resistance models.
- For analysis of structural members or smaller structures, more detailed FE models can be used. Here, shell elements can be used to build up sections consisting of parts with plate response, while continuum elements are needed in case of solid sections. In some cases, the model can be simplified to two dimensions assuming plane stress or plane strain. On this level of detailing, also failures due to e.g. local buckling in steel girders and shear failure in reinforced concrete members can be reflected.
- To reflect e.g. anchorage or detailed crack pattern in reinforced concrete members or details, the bond between the reinforcement and surrounding concrete must be included. A predefined bond-slip relation according to fib (2013) can be used to model the interaction along the reinforcement bars. To include the splitting effect and reflecting the influence of confinement and reinforcement yielding on the bond properties, a 3D bond model according to e.g. Lundgren (1999) is needed.

In case all possible failure modes are not reflected by the non-linear analysis, on the chosen level of detailing, these failure modes need to be checked by separate resistance models according to e.g. Eurocodes. The fib Model Code for Concrete Structures 2010, fib (2013), provides resistance models on different levels of detailing. This enables more sophisticated resistance models for e.g. shear and punching to be used together with the enhanced non-linear system analysis.

Non-linear FE analyses are used in a more enhanced level of assessment. On this level testing and monitoring can provide relevant input data concerning actual loads, in-situ material properties and actual behaviour of the bridge, and the bridge model can be updated based on on-site measurements, Schlune *et al.* (2008), ASCE (2011).

Structural assessment of ULS capacity based on non-linear FE analysis requires special considerations regarding the safety format. In a non-linear analysis reflecting both the overall stress distribution and local failure in the same model, the system and resistance analysis become integrated. The structural analysis becomes similar to an experimental test where the structure is subjected to increased loads until failure is reached. The verification of the structural safety is here made on the global level rather than by comparing load effects and resistance locally. The fib Model Code, fib (2013), provides safety formats for non-linear analysis, ranging from probabilistic method, over global resistance methods to the partial factor method. However, the model uncertainty given for complicated to model failure modes, like e.g. shear failure, may be un-conservative, Schlune (2011).

4.5.2 Degradation Modelling

Because deterioration and damage occurs along time, at the time of carrying out the assessment, the actual bridge characteristics and material properties should be considered. Therefore, the accurate modelling of the remaining strength, ductility, etc… is of main interest and the structural effects of the deterioration processes have to be taken into account. The main cause of material deterioration both in concrete and steel bridges is corrosion. Corrosion in steel bridges and performance profiles due to this process are the main issue of several deliverables of the MAINLINE project, as steel bridges was considered one of the main assets to be studied in the project. Therefore, corrosion in steel bridges will not be considered here as the subject as it is largely dealt with in other parts of the project. However, because corrosion in concrete bridges is also an important source of deterioration along life-cycle analysis, the way to model this degradation process in the advanced assessment of concrete bridges is considered in the present chapter.

Corrosion of reinforcement affects the structure in two ways: (a) volume expansion that generates splitting stresses in the concrete, which may crack and spall the concrete cover and affect the bond between reinforcement and concrete, and (b) area reduction and ductility change of the reinforcement bars. The effect on the bond can become dangerous if corrosion takes place in anchorage regions, such as splices, cut-off regions, or at end anchorages. The area reduction of the reinforcement bars will reduce both bending moment and shear capacity, and the effect on the ductility decreases the deformation capacity. All these effects reduce the safety of the structure; therefore they are important to understand and control.

A methodology to analyze the mechanical behavior and remaining load-carrying capacity of corroded reinforced concrete structures was proposed in Zandi Hanjari (2010). The methodology is shortly described in the following. It is based on the assumption that the usual method of structural analysis for concrete structures should be applied also to corroded reinforced concrete structures. The effect of corrosion is modelled as a change in the

geometry and material properties of the concrete, reinforcement and their interface through the following steps.

1. If corrosion caused the concrete to spall off, the effect on both the concrete cross-section and the cover loss can be taken into account by modifying the geometry used in the analysis. In compression regions where corrosion leads to cracking of concrete, lower strength and stiffness than for the virgin concrete should be assigned to cracked concrete. The behavior of concrete around corroded stirrups can be simulated by adapting lower tensile strength. The method of adjusting compressive and tensile strength of cracked concrete is described in Zandi Hanjari *et al.* (2011).
2. Reduction of the effective reinforcement area by both uniform and pitting corrosion is the most obvious effect to take into account. The actual area of a uniformly corroded bar can be calculated by assuming that corrosion has penetrated evenly around the bar. However, pitting corrosion affects the reinforcement locally; therefore, measurement or estimation of the pitting configuration is needed to be able to calculate the residual bar area, see e.g. Val and Melchers (1997). Finally, the ductility of corroded reinforcement can be calculated using practical models in which the residual ductility is confined to empirical correlations with area loss of the corroded reinforcement, see e.g. Cairns *et al.* (2005).
3. Corrosion affects the interaction of reinforcement and concrete. Therefore, the bond-slip relationship should be modified accordingly. The modification could be done according to the method proposed in Lundgren *et al.* (2012), where the level of corrosion corresponds to a certain amount of slip. This procedure can be applied to models at structural level where the bond-slip between the concrete and reinforcement is modelled by one-dimensional bond-slip relation. For simpler structural analysis models, such as beam-element analysis, where the bond-slip is not directly accounted for in the model, the anchorage length can be calculated by the procedure described in Lundgren *et al.* (2012). The basic 1D bond-slip differential equation is numerically solved in a Matlab-routine, resulting in an anchorage length required to anchor the yield force. Either the capacity of the reinforcement is then adjusted in the anchorage region, or the anchorage is checked manually. It could be noted that also a more advanced level of modelling bond is available; Lundgren (2002), Berra *et al.* (2003), and Zandi Hanjari *et al.* (2013) have used detailed finite element modeling to investigate the bond mechanism for corroded bars in concrete, in particular the effect of splitting stresses induced in the concrete by the volume increase of the corrosion products. However, this type of detailed three-dimensional (3D) modeling of the region around all the reinforcement bars is today mainly suitable for research purposes, as it is considered impractical for analysis of complete structures. It could also be noted that while earlier research almost solely has treated accelerated corrosion, recent results presented in Tahershamsi (2013) indicate that the reduction in bond capacity was smaller for naturally corroded specimens; thus, it appears to be safe to apply methods developed and verified for accelerated corrosion.

The methodology is exemplified in an assessment of “Gröndalsviadukten”, a bridge in Stockholm built in 1967 as presented in deliverable D1.3 of MAINLINE project.

4.6 Proof load testing

The evaluation of structures requires information related to its properties and real boundary conditions. This information cannot always be known with the desired accuracy, especially in existing structures, among other reasons because of incomplete documentation, unknown effects due to deterioration and uncertainty in the modelling of the structure. In these cases, the information can be obtained by non-destructive testing or partially destructive of the constituent materials and accurate measurements of the geometry of the existing structure. Even in extreme cases the structural safety can not be determined by analytical means, so that in certain cases it is beneficial to determine the structural safety through the execution of a load test in situ.

The non-destructive load tests on real structures are considered as one of the best methods to assess the carrying capacity, because the concept is intuitively acceptable and the results are conclusive.

The load test tests are an alternative or a complement to the assessment of bridges through analytical calculations. A proof load test evaluates the ability of the bridge to support its own weight and dead load, plus a certain overload. In order to obtain an adequate safety margin, against a possible overload, the bridge must be tested placing a greater load than the expected service load.

In many cases, the cost derived from the replacement of a structure can be avoided by testing the safety of the structure through a load test and, additionally, based on the results, the maintenance and repair of the structure can be more efficiently planned.

It is appropriate to carry out a proof load test if:

- The analytical methods produce an unsatisfactory load value of service, or
- The analytical method is difficult to perform due to the deterioration of the structure or the deficiency or absence of the necessary information for its application (drawings, material properties, etc.).

It is worth noting that a number of countries (e.g. UK) specifically recommend that proof load testing (i.e. load testing to the design or assessment load or above) should not be undertaken. This is because if the test is not properly performed and controlled, due to the high level of load in the bridge, a possible cracking or damage could be produced. For this reason, proof load is limited by the elastic behaviour of the materials, never going beyond this limit.

During the performance of the load test it is necessary to take several precautions, such as good planning, gradual implementation of the test load and control of the responses of the structure in order not to exceed the elastic limit of the material.

In general, the load test assays are performed to evaluate the load capacity of existing bridges, whose test loads are the maximum that the bridge can withstand without damage. Nevertheless the load proof does not necessarily have to be the top resisted by the bridge, but it must be of such magnitude that allows classifying the bridge as safe.

Since bridges are subjected to very high loads during the proof load tests, there is always the possibility that the bridge can permanently be damaged. However, this scenario is extremely unlikely. This can be controlled by a proper monitoring, using for instance acoustic emission among other techniques (Olaszek et al. 2010). In fact, acoustic emission has the advantage when compared to other measuring techniques that it may anticipate any malfunction or

damage of the bridge, by measuring a warning increment of the number of hints or a decrease on the parameter “b” in a b-value analysis (ElBatanouny et al. 2014). It is observed that the b-value reaches its minimum near the peak load and reaches maximum during micro cracking while it tends to decrease when micro cracks coalesce and start forming major and visible cracks (Vidya-Sagar et al. 2012). A complete description of the application of AE to an existing real bridge is presented in ARCHES-D16. It is shown there how the AE monitoring is an excellent tool to follow the response of the bridge to the increasing load and provides a robust criteria to stop the loading process (Olaszek et al. 2010).

The use of piezoceramic transducers (Song et al. 2007) has been proved also as a suitable monitoring to detect the existence of cracks and their growth in concrete, much earlier than other monitoring techniques (LVDT, visual inspection).

Despite some countries specifically do not allow to use proof load tests, other countries recognize them as alternative and complementary assessment tools. For instance in Canada, CAN/CSA-S6-06 gives guidance on bridge load testing. In general two types of tests are considered, static and dynamic. The static load tests are meant to capture the overall structural behaviour of more complex bridges and confirm the load distribution between their components and members. The dynamic load tests are performed to establish dynamic characteristics and behaviour of bridge structures. In particular, the dynamic load tests might be used to determine bridge specific dynamic amplification factors, which can be further considered in the load carrying capacity evaluation. Furthermore, in CAN/CSA-S6-06, proof loading is considered as an alternative method of load rating for bridges which cannot be accurately evaluated by analysis, or if the structural response of a bridge to live loads is questionable. A condition inspection and preliminary load rating of critical components in the bridge must be carried out prior to any load test. If during the test, the measured change in bridge behaviour is indicative of nonlinear load-deformations and inelastic strains, then the test must be immediately stopped and the maximum applied load may be assumed to be representative of the ultimate bridge capacity. When there is no apparent damage or change in the bridge behaviour during the test, but the test is stopped due to limitation of the test equipment, then the bridge capacity may be determined by an extrapolation of the test results. The extrapolation is based on comparing estimated initial strains due to the dead load and an acceptable level of maximum strain.

In the USA, the AASHTO LRFR (2003) also provides a reliability-based method for including the results of proof load testing during the safety evaluation of bridges and encourages the direct use of reliability methods in the rating of special bridges as compared to using the specified reliability-calibrated load factors.

The Code DAfSTb-Richtlinie (DAfStb, 2006-09) issued by the German Committee on reinforced concrete provides guidance on the proof-loading testing of reinforced concrete buildings.

Proof-load testing is used to update the information about the condition and actual capacity of the bridge based on the fact that the bridge has survived an external load that is perfectly known. However, many bridges that have never seen a load test are perfectly operating under normal traffic. Surviving a service load history that is stochastic in nature provides evidence of strength that may be comparable to what might be learned from a proof load test. This is what is called as service-proven bridge. A proof load test enables the lower tail of the resistance distribution to be truncated at the level of the maximum load carried. For a service-proven bridge, the magnitude of the maximum load carried is un-known, however, it can be determined statistically by using weigh-in-motion data as presented in Wang et al. 2011.

5. Strengthening of Structures

5.1 General

The purpose with this guideline is to assist the railway owners when deciding necessary strengthening measures for railway bridges of concrete, steel or masonry. In addition also possible strengthening measures for the subsoil are discussed. When a structure is strengthened, this is usually done in the ultimate limit state (ULS). However, many of the strengthening methods that are described in this document will also be applicable when measures are needed in the serviceability limit state (SLS), for example decreased crack sizes for concrete structures or increased stiffness for structural components. To compile all work from MAINLINE and Sustainable Bridges (2007) in a guideline, a new idea by using what we denote a “Graphical Index (GI)” has been used. The GI takes the standpoint in a structure or a structural member. The reason for strengthening is highlighted in a figure and methods through a Method Description for each method to solve the problem are referred to. In addition to this, Case Studies are connected to the method when possible. It needs to be stressed that a purpose has been to create a live document that should be easy to upgrade and that it also should be possible to add new components to the guideline. A new component could for example be design guidance or specific guidance for complicated production issues.

The guideline is divided into a Graphical Index document, Section 5.5 below, Method Description documents, Appendix E, and Case Study documents, Appendix F. Method descriptions give detailed description of the strengthening method referred to, equipment used, benefits and drawbacks and a cost estimate of the method. In the case studies different field applications of the method descriptions are presented. Further descriptions and case studies are given in SB-STR (2007) and Täljsten et al (2006, 2011).

The method description and the case studies follow a template and it is easy to add new methods or case studies to the guideline following these templates. However, when adding new strengthening methods it is suggested that an expert within that area is consulted. The intention is also that when a strengthening project is finished the Template Case Study, see Appendix F, shall be filled in and a new Case Study can be added to the guideline (database). In the guideline references also are made to other documents which for example could mean a code, a research report or other useful documents that we recommend the reader to study. In the next section some useful definitions are explained.

5.2 Aim and Limitations

One aim with the guideline is to highlight strengthening methods that are environmental friendly, not disturbing the ongoing traffic and at the same time being cost competitive. A second aim is to create a guideline easy to handle for the final users and possible to upgrade over time.

The guideline has been limited to methods and strengthening systems known to the authors. Methods that can be considered traditional or methods that can be considered well known to railway Infrastructure Managers (IM) have not been discussed in depth in the guideline. However, it would be quite easy to add additional methods in more detail if one so wish.

Furthermore the reason for strengthening is only discussed briefly, since the background for repair and strengthening often varies. The main focus is on bridges as they are structures where strengthening has a high effect. Tunnels are seldom in need of strengthening in order to carry higher loads; it is more often a question of enlarging the cross section. For track it.is

usually better to change deficient rail or sleepers than to try to strengthen them. Fatigue is treated in Mahal (2015).

5.3 Definitions

5.3.1 General

In general when speaking about repair and strengthening of structures the following definitions are normally used:

Maintenance: To keep a structure performance at its original level

Repair: To upgrade a structure performance to its original level

Upgrading: To increase the performance of a structure

Performance is related to durability, load carrying capacity, aesthetics and the serviceability of a structure. Performance is often related to a minimum safety level.

5.3.2 Bridge components – glossary of terms

Bridge: A bridge is a structure spanning and providing passage over a river, chasm, traffic intersection area, fjord, inlet or other physically obstacles and with a span length equal or exceeding a certain distance. This distance is defined by national authorities and is usually in the range of 2-6 m, Brime Report, Woodward (2001). A bridge may also be divided into the superstructure, the substructure and the ground. These parts are shown in Figure 5.1.

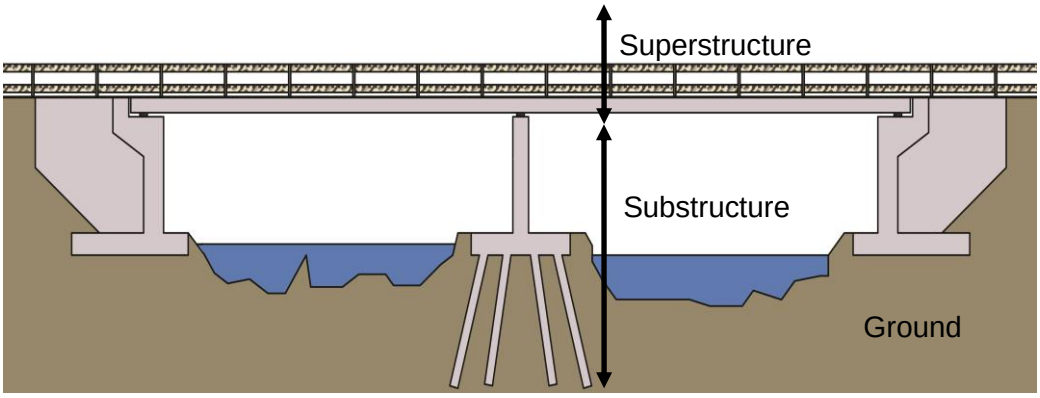


Figure 5-1. Principal parts of a bridge

The superstructure carries the traffic loads together with its self-weight to the substructure through the bearings. Examples of structural elements belonging to the superstructure can be slab, girder and deck.

The substructure carries the load from the superstructure together with its self-weight through the foundations to the supporting ground. Examples of structural elements which the substructure is divided into are abutments, piers, columns, towers etc.

The ground in this context is that element which takes the loads from the foundation and the surrounding area. Examples of the elements which the ground is divided into are embankment, in-situ soil, fill etc.

In this report, “longitudinal” will be used for the direction parallel to the rail track and “transverse” will be used for the direction transverse to the longitudinal direction either horizontally or vertical as appropriate in different situations.

5.4 Repair and strengthening of structures

5.4.1 General overview

To repair or strengthen existing structures is a complicated task. This is mainly due to the fact that the conditions are already set and that it often can be complicated to decide the underlying reason for the strengthening need. In addition to this strengthening is mostly carried out for improved load carrying capacity in the ultimate limit state but a structure is almost only loaded in the service limit state, which here also includes fatigue and durability limit states. This means that the strengthening needs and design must be based on theoretical assumptions that might be difficult to verify. Despite this, there is a quite good understanding how structures behave with different strengthening measures. In this section a general discussion regarding repair and strengthening philosophy is made, discussion the connection to safety.

In Appendix A a more detailed descriptions of traditional strengthening methods are presented.

In Figure 5.2 the general complexity repairing or strengthening a structure is explained. First assume that we have a concrete structure in need of strengthening. Furthermore, in this simplified example, we assume that we have a client that owns the structure, a consultant that provides the measures and a contractor that carries out the strengthening work. Codes and standards must be followed, and in some cases universities of testing institutes take and test material samples from the structure.

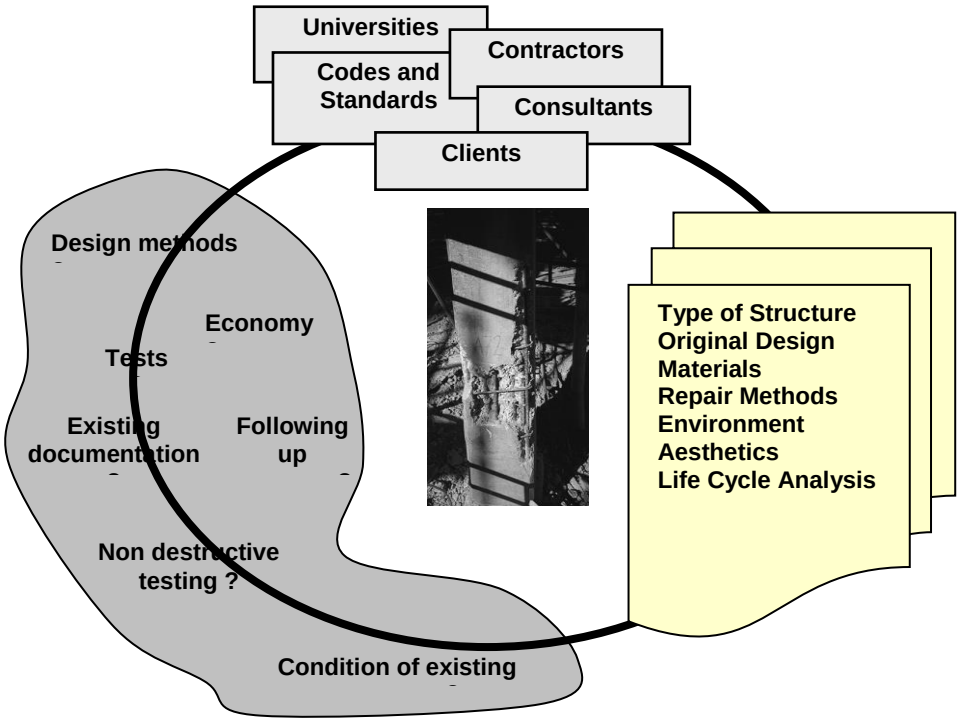


Figure 5-2. Complexity when repairing or strengthening a structure

The type of structure, and the building component, that is going to be strengthened must be considered. Is it a bridge? A column? A beam etc.? Different component and different structures need different measures.

To consider the original design is always important, in particular for older structures that used other guidelines and codes than today. The original design form the base in the strengthening need and here also all existing documentation and history for the structure should be considered when applicable. The next step is to consider the material in the structure and the material that are added after strengthening. Is the old and new material compatible or not? For example must composite action be obtained to transfer the forces from the structure to the strengthening? Preferably we should be able to choose between different repair and strengthening methods and hence choose the most suitable one for the structure or component studied. Important is also to consider environmental issues, are we using the most possible environmental friendly products? Furthermore, the aesthetics and life cycle aspects must also be considered. Do we obtain a better appearance after strengthening and most important do we prolong the life and performance of the structure. Wrong choice of method might decrease the life.

For all method the cost must be considered and the cost should take in consideration the desired function and the remaining expected life of the structure.

In complicated cases tests may be needed and systems to follow up the strengthening structure over time introduced. Often these programs can be a combination of measurements and physical inspections, where the physical inspections are carried out more often the first years after completion.

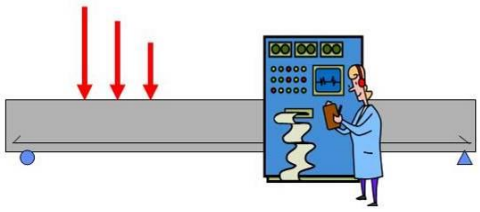
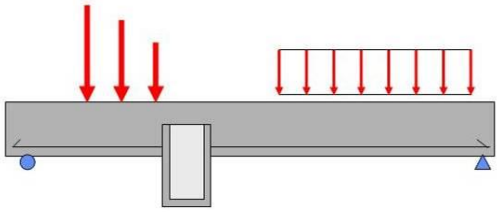
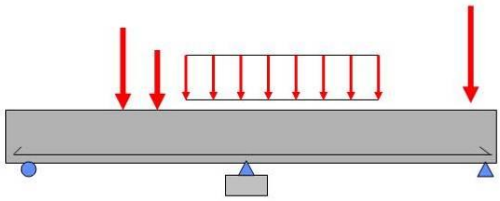
Most complicated is always to decide the condition of the existing structure and for complicated cases it is very important that a proper assessment is carried out. This assessment should be a combination of testing, site investigations and theoretical calculations.

Thus, it is important to choose the most suitable method when a structure or component is going to be upgraded. Which method that is most suitable will vary from object to object. If upgrading in general is discussed and in particular for concrete structures the principles for upgrading is shown in Table 5.1.

Often most economical is to carry out what can be denoted a refined calculation where more detailed calculation tools are used, for example FE-analysis and considerations to real material data and structural dimensions are taken. However, other methods to upgrade a structure might be increased cross section, which is a common method for concrete structures but can also be used for steel structures by welding additional steel parts to the structural member. Sometimes it might be possible to change the static system for the structure, transfer the loads into other parts that then can take up the new loading. For concrete structure, but also for metallic structure, external prestressing can be a suitable method to increase the load carrying capacity. Here a axial, positive or negative, load is introduced to the structure by prestressing. Building components can also be upgraded by external bonding of composite materials. Concrete as well as metallic, masonry and timber have been strengthening by this method. The method is considered relatively new in the building industry even though it has been used during the last decade quite frequently.

This method is also the focus in the work presented here. Primarily since it was found that the knowledge and experience about traditional methods are high, but also due to the reason that the knowledge and experience regarding FRP strengthening of railway bridges is small.

Table 5-1. General upgrading methods

	
Refined calculations	Increased cross section
	
Change of static system	External prestressing
	
External bonding with FRP	

Nevertheless, to be able to choose the most suitable strengthening method for a railway bridge a clear structure should be followed. It was fund that a structured approach for strengthening applications was missing. Therefore, a large amount of work was placed on developing a structured methodology for strengthening of existing railway bridges. This structure has been denoted “Graphical Index”. This is discussed thoroughly in the coming chapters in this report. To the graphical index, Method Descriptions and Cases Studies are connected.

However, it should also to be mentioned that the approach is intended to be a living document which was initiated in Sustainable Bridges (2007) and is now continued in MAINLINE. Its value for the railway owners is very much dependent on the information that is entered into it.

5.5 Graphical index

5.5.1 Introduction

This section serves as a help to graphically navigate in the wide selection of repair and strengthening methods covered. The most common bridge types are listed and sorted into three categories; reinforced concrete bridges, metallic bridges and masonry arch bridges. Each category is then further divided into types, i.e. beams, truss, box girder, arches, and so on as applicable for each category of bridges. However, not all bridges will be covered by this approach so typical structural elements such as beams, slabs and columns etc. are also presented. For composite bridges, e.g. concrete slab on steel girders, guidance from both reinforced concrete and steel bridges may be applicable. For each bridge type and structural element, a sketch is presented in figures. Locations for common problems are marked with a letter and in connection to the sketch explanations for each letter are given, i.e. in supporting tables. Explanations include; summary of problem description, and references to further information within the report, possible solution and best practise. In addition to bridges, repair and strengthening needs and possible methods to solve this are also given for the sub soil and foundations.

5.5.2 Reinforced Concrete Bridges

General

Relatively many concrete structures have reserve in compression capacity and the amount of existing steel reinforcement limits the load carrying capacity. In cases when compressive strength limits the capacity, it is not possible to give general suggestions for improvement. In this section possible strengthening need will be given for; girder, trough, box and arch bridges.

Bridges with T-Beams

In a beam bridge, the beams constitute the main carrying structural element. They have to carry the loads from the traffic and the weight of the bridge deck as well as its self weight and transfer the vertical and horizontal forces down to the substructure of the bridge. The beam bridge type of structure includes in situ casting and prefabricated beams and girders. The concrete beam bridge is constructed of two or more beams. The beams may be of T-section, rectangular or in other shapes. In Figure 5.3 a beam bridge is shown with possible load carrying limitations marked. Methods for remedying the shortcomings are listed in Tables 5.2 to 5.6.

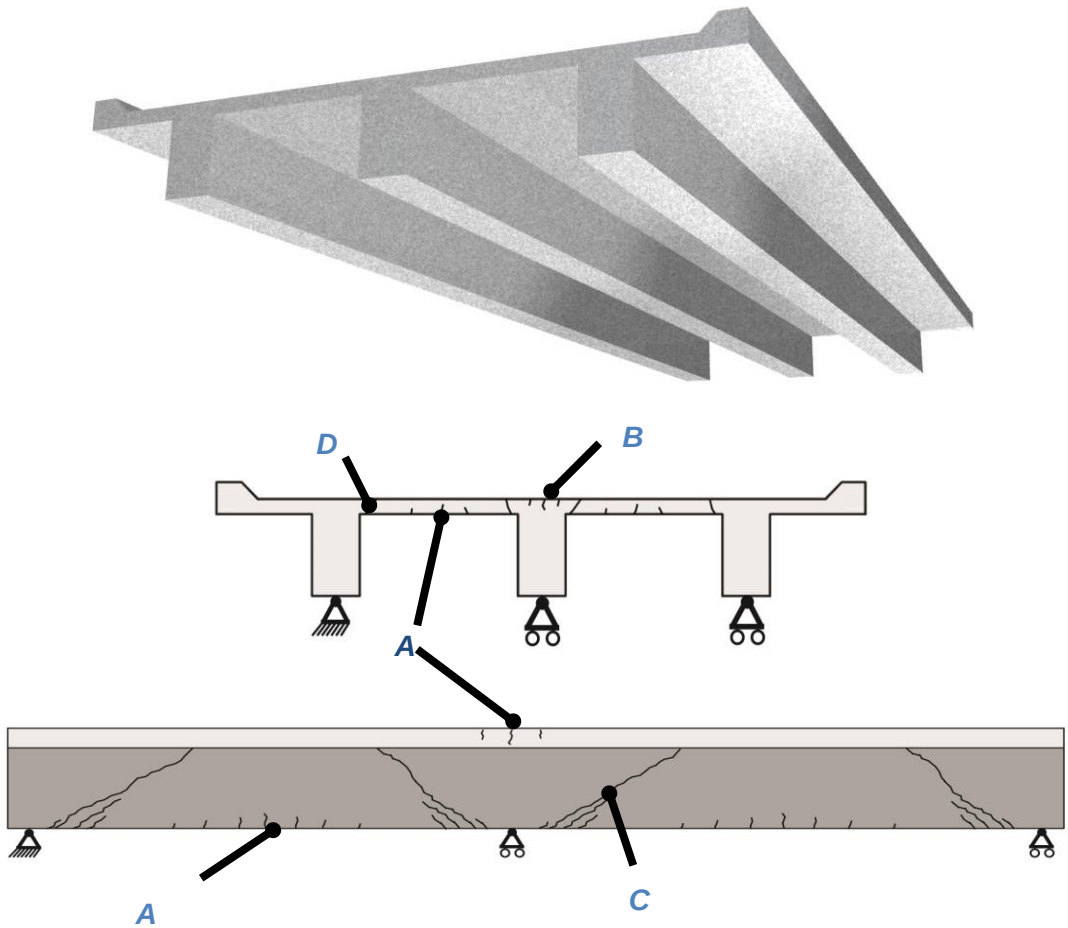


Figure 5-3. Beam Bridge with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; B - Deficient flexural bearing capacity in areas hard to access; C - Deficient shear bearing capacity in beams; D - Deficient shear bearing capacity in slabs. SB-STR (2007).

Trough Bridge

The trough bridge, shown in Figure 5.4, can be found in numerous alternative designs. Typical is that two main girders, sometimes with a small flange at top, are connected with a slab at the lower part forming a trough. The trough is normally filled with ballast with one or two railway tracks at the same level as the upper part of the girders. The main girders can be in different size and shape with aligned or vertical sides. Bridges are often skewed.

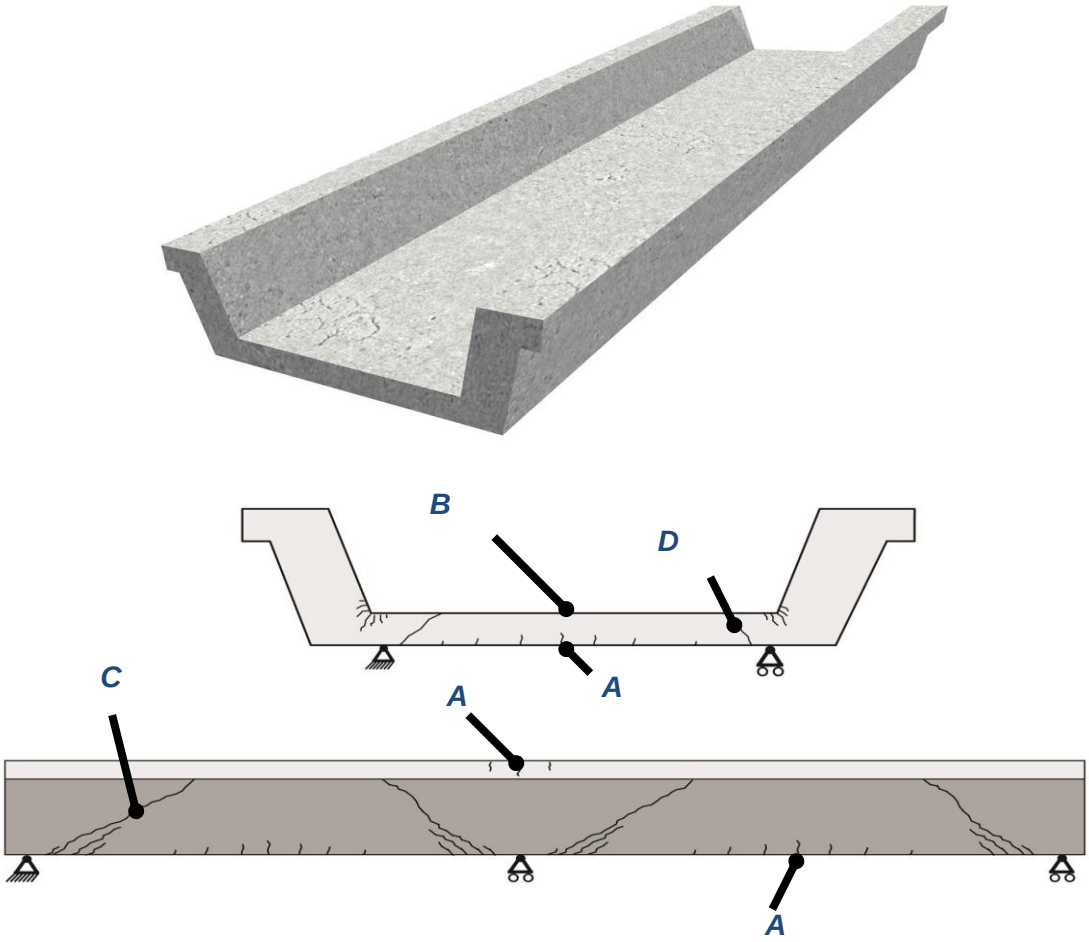


Figure 5-3. Reinforced concrete trough bridge with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; B - Deficient flexural bearing capacity in areas hard to access; C - Deficient shear bearing capacity in beams; D - Deficient shear bearing capacity in slabs. SB-STR (2007).

Box Girder Bridge

Box girder is so named because of their appearance since they look like a box. Compared to normal beams the bridge deck of a box girder is identical to the top flange, the walls form the web and the bottom plate is similar to the bottom flange. The box girder bridge, shown in Figure 5.5, is often used for spans in the longer region. In such cases, the bridge carries a high amount of dead-load. Box girder bridges are often post tensioned and carry normal two railway lines or more.

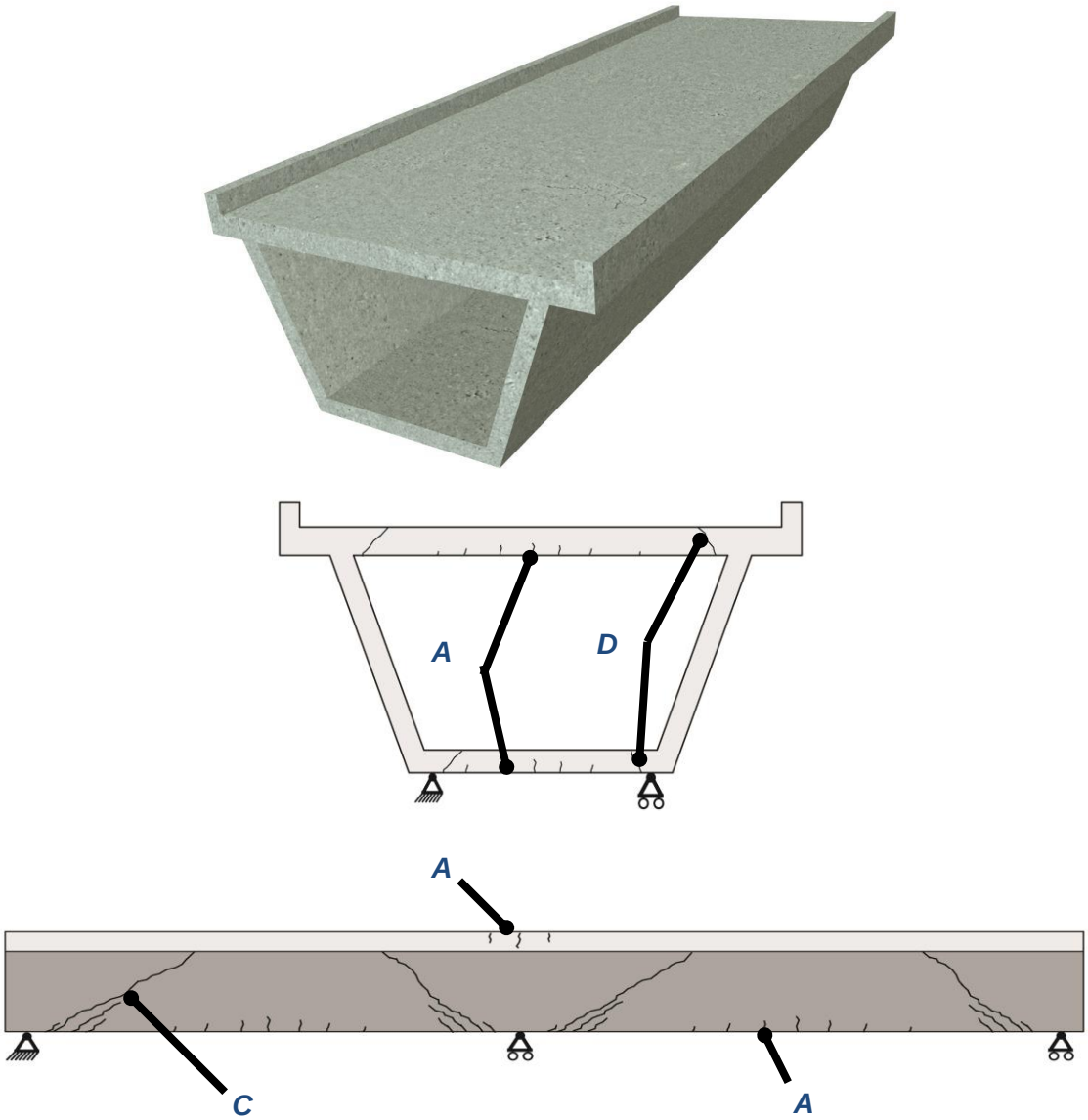


Figure 5-4. Reinforced concrete box girder bridge bridge with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; C - Deficient shear bearing capacity in beams; D - Deficient shear bearing capacity in slabs.SB-STR (2007).

Concrete arch bridge

The type of structure termed arch includes open and closed spandrel arch bridges as well as earth filled arch bridges. The bridge deck can be either above, between or underneath the arches. The concrete arch shown in Figure 5.6 can be found in several designs. Most arches are designed to carry compressive forces. However the arches often contain steel reinforcement and with the desired loads of today, bending and tensile strains may be introduced in the arch. This means that arches once designed primarily for compressive forces, may have sufficient shear or flexural capacity.

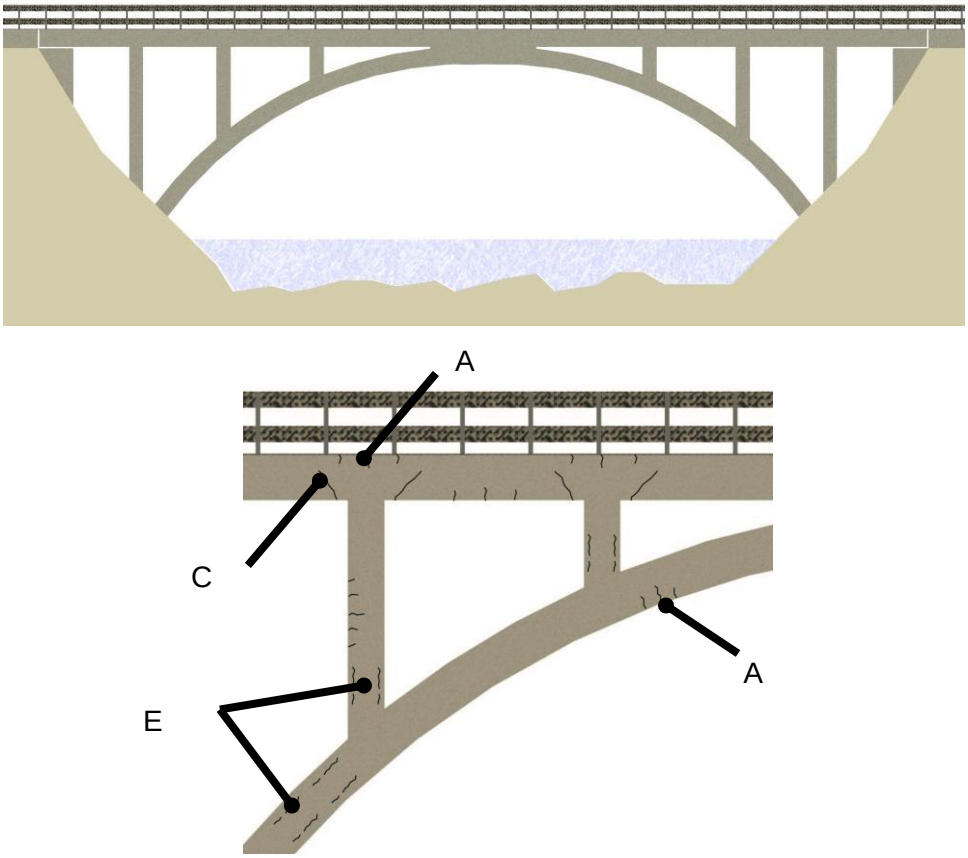


Figure 5-5. Concrete arch bridge with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; C - Deficient shear bearing capacity in beams; E – Deficient bearing capacity in columns and arches. SB-STR (2007).

Concrete structural components

There exist other bridges of reinforced concrete as well, not presented in this report. Instead of aiming to present all possible concrete bridges, structural components used in bridges will be presented. Figure 5.7 shows a concrete column subjected to high compressive forces.

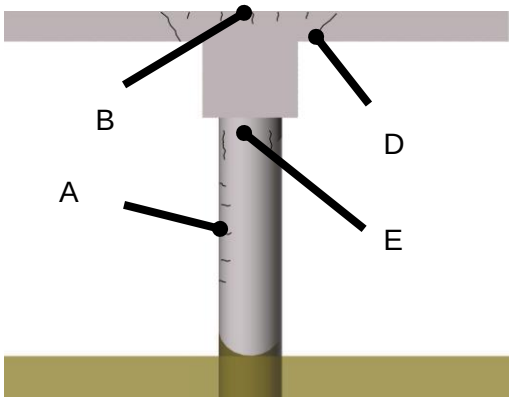


Figure 5-6. Concrete column and supported slab with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; B - Deficient flexural bearing capacity in areas hard access; D - Deficient shear bearing capacity in slabs; E – Deficient bearing capacity in columns and arches. SB-STR (2007).

Figure 5.8 shows a concrete beam that can be loaded in any direction. However, most common for beam elements is that the element is loaded in a combination of shear and flexure.

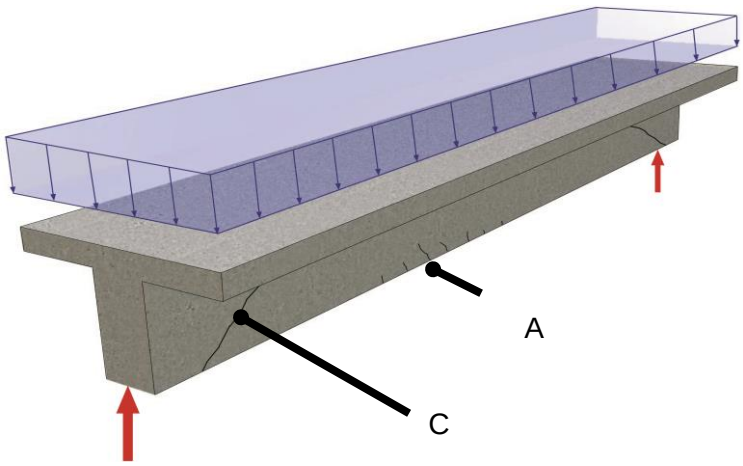


Figure 5-7 Typical concrete beam with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; C - Deficient shear bearing capacity in beams; D - Deficient shear bearing capacity in slabs. SB-STR (2007).

Slabs, Figure 5.9, may be supported in several ways. I.e., slabs may have supports along edges as when fixed between two beams or in a point as when placed above a column.

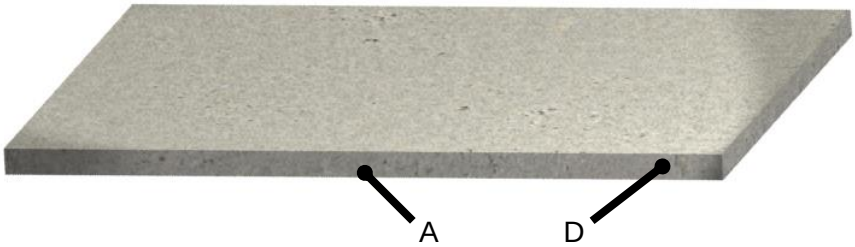


Figure 5-8. Concrete slab with indication of possible areas for upgrading. A – Deficient flexural bearing capacity; D - Deficient shear bearing capacity in slabs. SB-STR (2007).

5.5.3 Deficiencies in Concrete Beams and Structural Components

For the above shown concrete bridges and concrete structural components letters showing possible deficiencies have been added to the figures. These letters denotes different types of codes related to typical problems that can result in strengthening measures. In Table 5.2, for example, typical flexural cracks and methods how to take care of that specific problem is suggested and in Table 5.3 methods to strengthen structures with flexural cracking in regions that cannot be directly accessed is suggested. Other references in the tables refer to SB 6.2 (2007), SB6.3 (2007) and SB 6.4 (2007).

Table 5-2. Reinforced concrete, code A

Problem	Deficient **flexural bearing capacity**. Cracks starting at the bottom or top of the beam, propagating towards neutral layer, may be an indication that the member needs strengthening in flexure. Calculations could also show the need for strengthening. See Appendix A-G.		
Strengthening method	Method Description	Case Study	Other references
External CFRP Plate	MD001	CS001	D6.2, D6.4
External CFRP Sheet	MD002	CS002	D6.2, D6.4
MBC	MD003		D6.2
NSMR	MD004	CS004	D6.2, D6.4
External prestressing, longitudinal	MD005	CS005	D6.2
Increased reinforced cross section			D6.2

Table 5-3. Reinforced concrete, code B

Problem	Deficient **flexural bearing capacity** in regions which **cannot be directly accessed**. The structural problem is similar to other flexural deficiencies but the number of solutions is limited. See Appendix A-G		
Strengthening method	Method Description	Case Study	Other references
External prestressing Internal steel/CFRP stays	MD005	CS005	D6.2

In Table 5.4 possible measures to strengthen concrete structures for shear are given.

Table 5-4. Reinforced concrete, code C

Problem	Deficient **shear** bearing capacity of **beam**-like structural elements. The problem may be addressed by calculations or inclined cracks in middle of the beams height, close to support or where amount of shear reinforcement changes. See Appendix A-G .		
Strengthening method	Method Description	Case Study	Other references
External CFRP Plate	MD001	CS001	D6.2, D6.4
External CFRP Sheet	MD002	CS002	D6.2, D6.4
MBC	MD003		D6.2
NSMR	MD004		D6.2, D6.4
External prestressing, longitudinal	MD005	CS005	D6.2
External prestressing, transverse	MD005		D6.2
New stirrups and concrete casting*			
Internal steel/CFRP stays*			
Stiching*			
Fibre reinforced shotcrete			

* The method exists, even though not further studied within the project.

Table 5-5. Reinforced concrete, code D

Problem	Deficient s**hear** bearing capacity of **slab**-like structural elements. Problems with low shear bearing capacity of slabs are likely near supports, i.e. at connections to other structural members such as beams or columns. See Appendix A-G		
Strengthening method	Method Description	Case Study	Other references
Internal steel/CFRP stirrups*			
Internal prestressing	MD005		D6.2
External prestressing	MD005	CS005	D6.2
Increase of load transfer area*			

* The method exists, even though not further studied within the project.

Table 5-6. Reinforced concrete, code E

Problem	Deficient **compression bearing capacity in a column**. In addition to high compressive forces, columns are also likely to be subjected to bending and shear forces. Strengthening methods for bending and shear are presented for code A – D. See Appendix A-G..		
Strengthening method	Method Description	Case Study	Other references
External CFRP	MD002		D6.2, D6.4
Steel jacket			
Increased reinforced cross section			

5.5.4 Metallic Bridges

General

Metallic bridges can be made of cast iron, wrought iron, steel, stainless steel or aluminium. They can have several different designs such as girder, truss and box girder bridges. In addition to this, metallic bridges can be found in almost any design with different material qualities for different structural members within the same structure.

Metallic Truss Bridge

The superstructure of a truss bridge commonly consists of two vertical trusses on each side of the track. Track can also be located above truss beams. These trusses are the primary structural elements of the bridge and they consist of many members, such as top and bottom elements, diagonal and vertical struts, and end struts. These members form the truss and are mainly subjected to tensile or compressive forces. Horizontal stability is given by a horizontal truss at the top or bottom of the bridge. The horizontal truss gives bearing capacity for horizontal loads from wind and horizontal accelerations of the train. All truss members are mutually connected by bolts or rivets.

The truss walls have the same function in a truss bridge as a beam in a beam bridge. The top and bottom members of a truss are similar to the top and bottom flange of a beam, and the end, vertical and diagonal struts are similar to the web of the beam. A schematic truss bridge is shown in Figure 5.10. This bridge type is often used for bridges that are built before 1950-ties and can be found for medium spans.

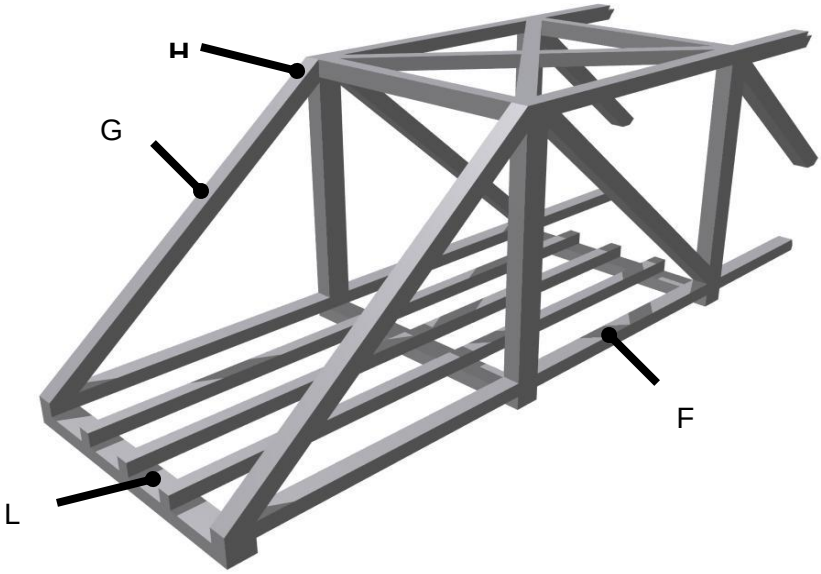


Figure 5-9. Metallic truss bridge with indication of possible areas for upgrading. F – global bearing capacity or stiffness; G – compression bearing capacity; H – joints including rivets; L – local load-bearing capacity

Metallic box girder bridge

Metallic box girders normally have a deck of concrete, and are then called composite bridges which are discussed later on, but entirely steel box girder do also exists. Figure 5.11 shows a box girder steel bridge.

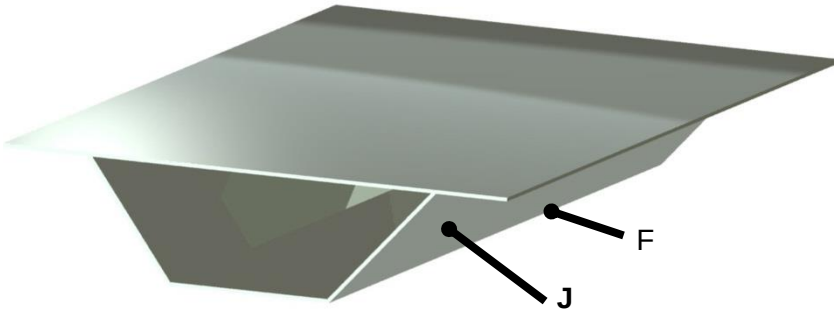


Figure 5-10. Metallic box girder bridge with indication of possible areas for upgrading. F – global bearing capacity or stiffness; J – shear bearing or buckling.

Metallic structural components

The most common metallic bridges are made of different compilation of welded, riveted or bolted beams. In Figures 5.12 and 5.13 a welded beam profile and a riveted beam are shown. Beams can be loaded in any way. However, a combination of shear and flexural loading dominates.

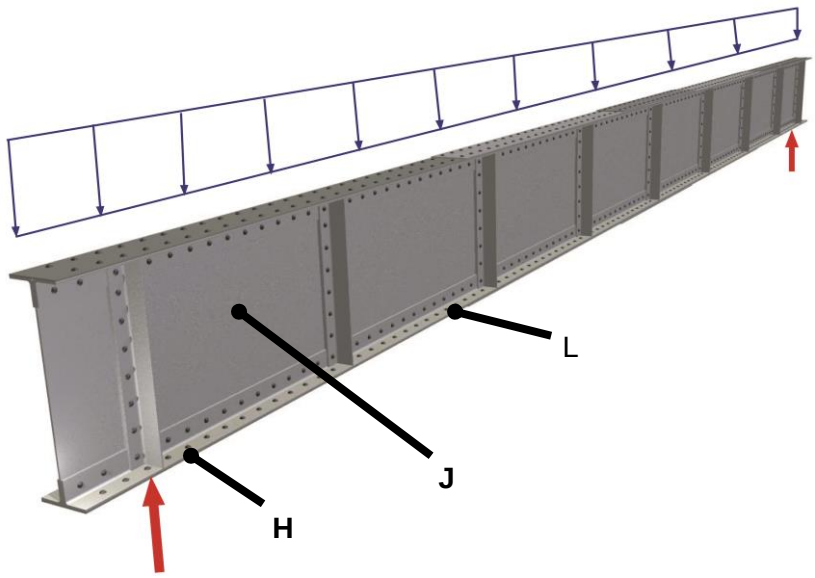


Figure 5-11. Welded and riveted metallic beam element with indication of possible areas for upgrading. H – joints including rivets; J – shear bearing or buckling; L – load-bearing capacity.

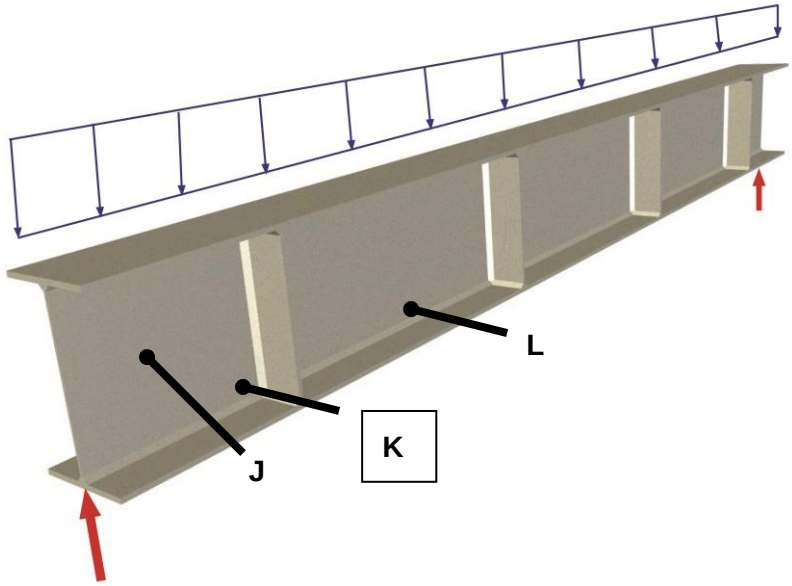


Figure 5-12. Welded and riveted metallic beam element with indication of possible areas for upgrading. J – shear bearing or buckling; K – stress concentrations; L – load-bearing capacity.

Table 5-7. Metallic structures, code F

Problem	Deficient **global bearing capacity or stiffness**. Deficient tension capacity in structural elements and joints. See Appendix A-G.		
Strengthening method	Method Description	Case Study	Other references
External prestressing, longitudinal	MD103		D6.2

Table 5-8. Metallic structures, code G

Problem	Deficient **compression bearing capacity** of beam-like elements. Relatively many structures are slender and have reserve in compression capacity hence global buckling limits the capacity. See Appendix A-G.		
Strengthening method	Method Description	Case Study	Other references
External CFRP Plate	MD101	CS101	D6.2, D6.4
External CFRP Sheet	MD102		D6.2, D6.4
Increase of cross section			
Change of structure, i.e. buckling length			

Table 5-9. Metallic structures, code H

Problem	Deficient bearing capacity of **joints including rivets**. Cross-sections are often changed in joints causing stress concentration which may lead to fatigue problems. See Appendix A-G.		
Strengthening technique	Method Description	Case Study	Other references
External prestressing, longitudinal tendons	MD103	CS03	D6.2
External prestressing, prestressed CFRP plate	MD104		
Replace of components, rivet, bolt, plate			

Table 5-10. Metallic structures, code J

Problem	Deficient **shear bearing or buckling capacity** of webs or flanges. See Appendix A-G.		
Strengthening technique	Method Description	Case Study	Other references
Longitudinal stiffeners			
Transverse stiffeners			
External CFRP	MD101	CS101	D6.2, D6.4

Table 5-11. Metallic structures, code K

Problem	In conjunction to welds **cross-sections are changed** in joints causing stress concentration which may lead to fatigue problems whith possible cracks. See Appendix A-G.		
Strengthening technique	Method Description	Case Study	Other references
Weld cracks			
Drilling stop holes			
External prestressing	MD103		D6.2
External CFRP	MD01		D6.2, D6.4

Table 5-12. Metallic structures, code L

Problem	Deficient **flexural load bearing capacity** often addressed by calculations. Insufficient stiffness can be also be solved by strengthening. See Appendix A-G.		
Strengthening technique	Method Description	Case Study	Other references
External prestressing	MD103	CS03	D6.2
External CFRP	MD101	CS101	D6.2, D6.4

5.5.5 Sub soil and foundations

Location of subsoil and example of foundation are given in Figure 5.14.

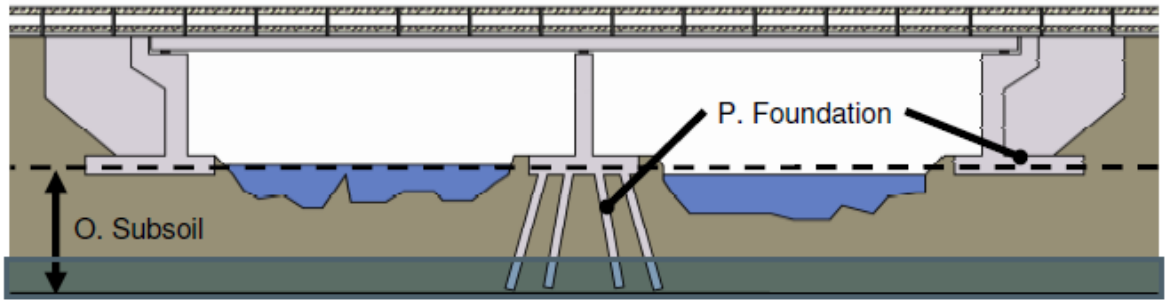


Figure 5-13. Subsoil and foundation with indication of possible areas for upgrading. O – subsoil; P – Foundations.

The subsoil conditions in the transition zones at the bridge abutments govern the applicable strengthening method. Low stability of embankment in transition zones might be a problem, see Figure 5.15.

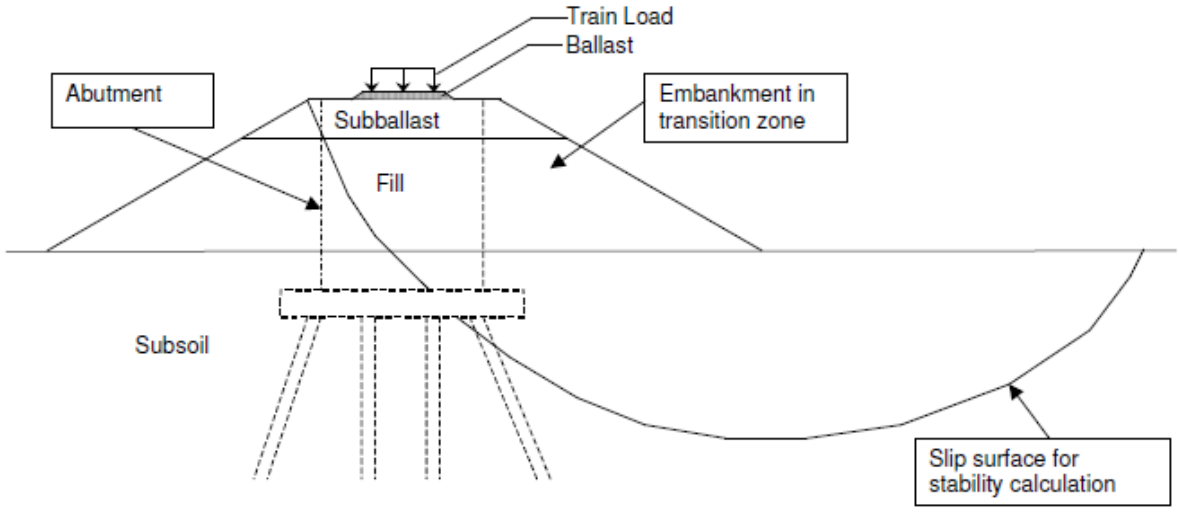


Figure 5-14. Principle for the analysis of stability of embankment in transition zones

Low stability of embankment in transition zones might be a problem, see Figure 5-15-. Regarding the slip surface for stability calculations refined methods may be necessary in deformation-softening clays, see Bernander (2011).

Depending on the subsoil conditions the foundation of bridge abutments and piers can be a shallow foundation or a deep foundation. At subsoil with high bearing capacity shallow foundation is normally used. At soil subsoil with poor bearing capacity deep foundations is used. The principle zones of improvement at shallow and deep foundation of the bridge abutments are shown in Figure 5.16 and Figure 5.17 respectively.

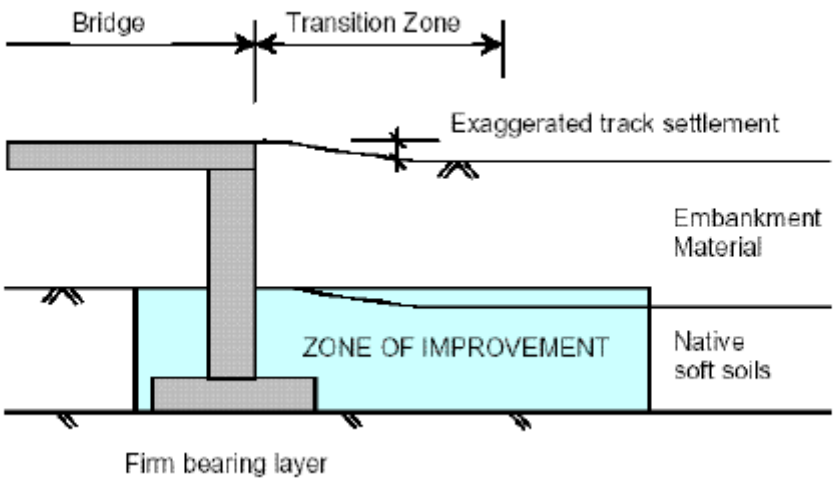


Figure 5-15. Principle sketch of zone of improvement at shallow foundation of bridge abutment

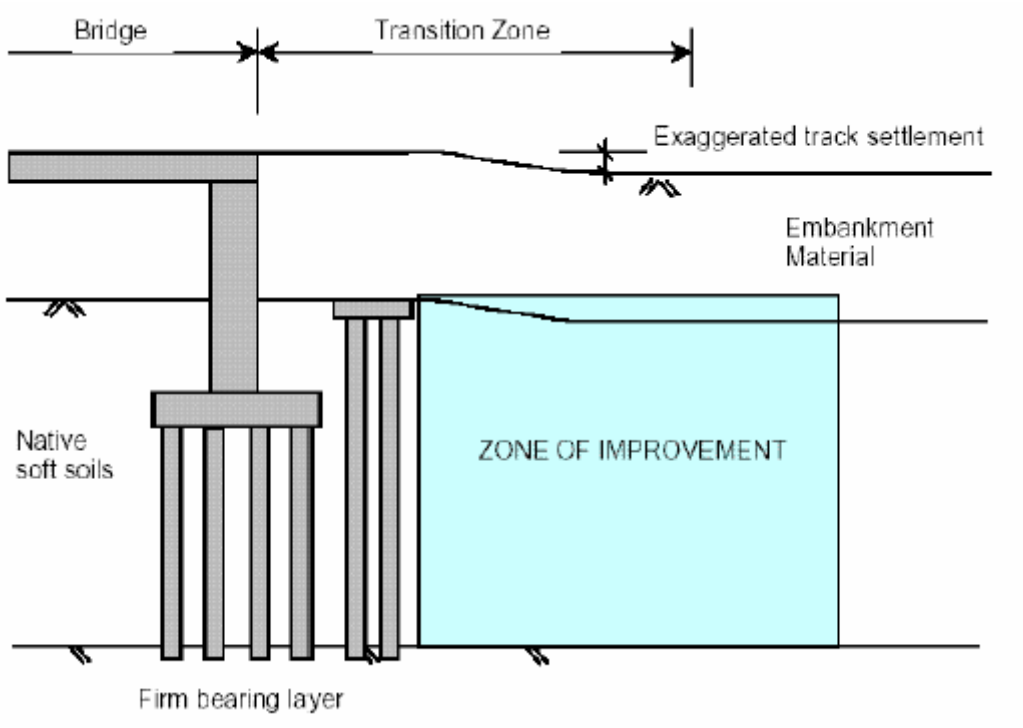


Figure 5-16. Principle sketch of zone of improvement at deep foundation of bridge abutment

In the Method Descriptions the applicable soils, advantages/disadvantages, standard/ guidelines etc. for the different methods are given. In Table 5.13, possible strengthening methods for sub soil are given

Table 5-13. Sub soil, code O. See Appendix A-G.

Problem	Settlements in subsoil layer or low stability of railway embankment with respect to the subsoil layer.

Strengthening method	Method Description	Case Study	Other references
Deep Mixing	MD301		D6.2
Jet Grouting	MD302		D6.2
Sheet Pile Walls/Stabilising Berms	MD303	CS303	D6.2
Compaction Grouting	MD304		D6.2
Embankment Piling	MD305		D6.2

Table 5-14 .. Possible strengthening methods for foundations. See appendix A-G.

Problem	Low bearing capacity of the foundation of bridge.		
Strengthening method	Method Description	Case Study	Other references
Jet Grouting	MD306		D6.2
Compaction Grouting	MD307		D6.2
Shaft Grouting and Base Grouting	MD308		D6.2

5.5.6 Example how to use graphical index

In this section a brief description how to use the graphical index is given. Assume that we have a concrete bridge as in Figure 5.18. The bridge consists of a superstructure built up by three reinforced concrete T-beams placed adjacent, which carries the track including ballast. During ordinary inspection aligned cracks have been found in the web of one of the girders close to the supports. It is then assumed that these cracks lead to the need of strengthening. The guideline is then used for finding a strengthening method in the following way:

First, it is clear the bridge is built from Reinforced concrete. The bridge is of the same type as the beam bridge presented in Figure 5.3. By studying the figure and corresponding tables for marked codes, the letter “C” is found as the code for the problem.

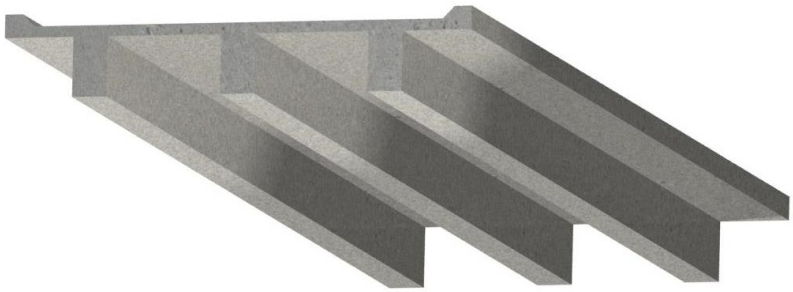
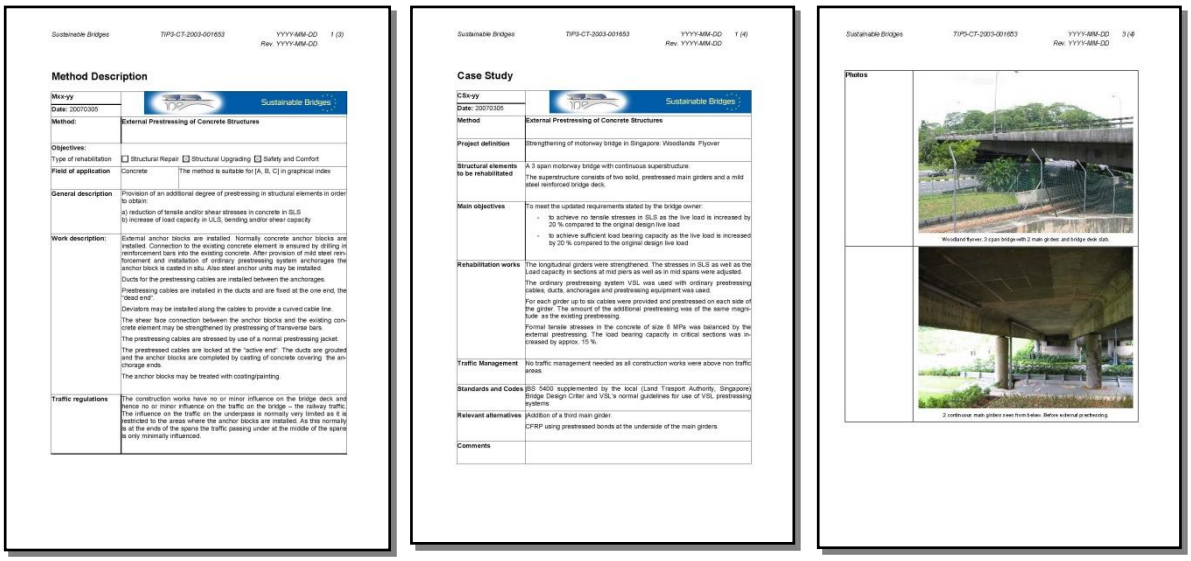


Figure. 5-17. Concrete T-girder bridge

Table 5.4, reproduced in Figure 5.19, gives the definition for code C. The found inclined cracks indicate deficient shear bearing capacity. The concrete has not crushed. Hence, additional shear bearing capacity might be achieved with additional reinforcement. Table 5.4 gives a variety of possible strengthening methods. Depending on experience, skill and knowledge each of the methods is further studied. Here the Method Description (MD) is then referred to, see Figure 5.19 for the different methods. Most suitable method should be chosen.

Problem	Deficient **shear** bearing capacity of **beam**-like structural elements. The problem may be addressed by calculations or inclined cracks in middle of the beams height, close to support or where amount of shear reinforcement changes. See Appendix A-G .			
Strengthening method		Method Description	Case Study	Other references
External CFRP Plate		MD001	CS001	D6.2, D6.4
External CFRP Sheet		MD002	CS002	D6.2, D6.4
MBC		MD003		D6.2
NSMR		MD004		D6.2, D6.4
External prestressing, longitudinal		MD005	CS005	D6.2
External prestressing, transverse		MD005		D6.2
New stirrups and concrete casting*				
Internal steel/CFRP stays*				
Stiching*				
Fibre reinforced shotcrete				

Figure 5-18. Example of different strengthening systems from Table 5.19.



Method Description Case Study Case Study cont.

Figure 5-19. Method Description and Case Study, See Appendix A-G and SB-STR (2007)

In this particular case, when the methods have been studied, it was found that “External CFRP Sheet, MD002” and “MBC, MD003” will be the possible solutions. However, when also the case studies were studied it was found that a similar problem have been solved by external CFRP Sheets and according to case study, “CS002, External CFRP Sheet”. Finally this method was found to be the most preferable strengthening method in this case.

The next step from here is to follow the procedures for design and production planning. How this can be done is not presented in this guideline, but can be found in national codes and in delivery SB 6.4 (2007). After a project is concluded it is strongly recommended that the template for “Case Study” is filled in and saved as a new case study for experience and future use.

6. Tunnels

6.1 Introduction

A general handbook on tunnels is Maidl et al (2013, 2014). Below some guidelines are given. More material can be found in ML-D1.3 (2014)

Railway tunnels are almost impossible to replace so they must be maintained on the assumption that they have a virtually infinite life. They can be either lined or unlined and have a variety of shapes as illustrated in Figure 6.1.

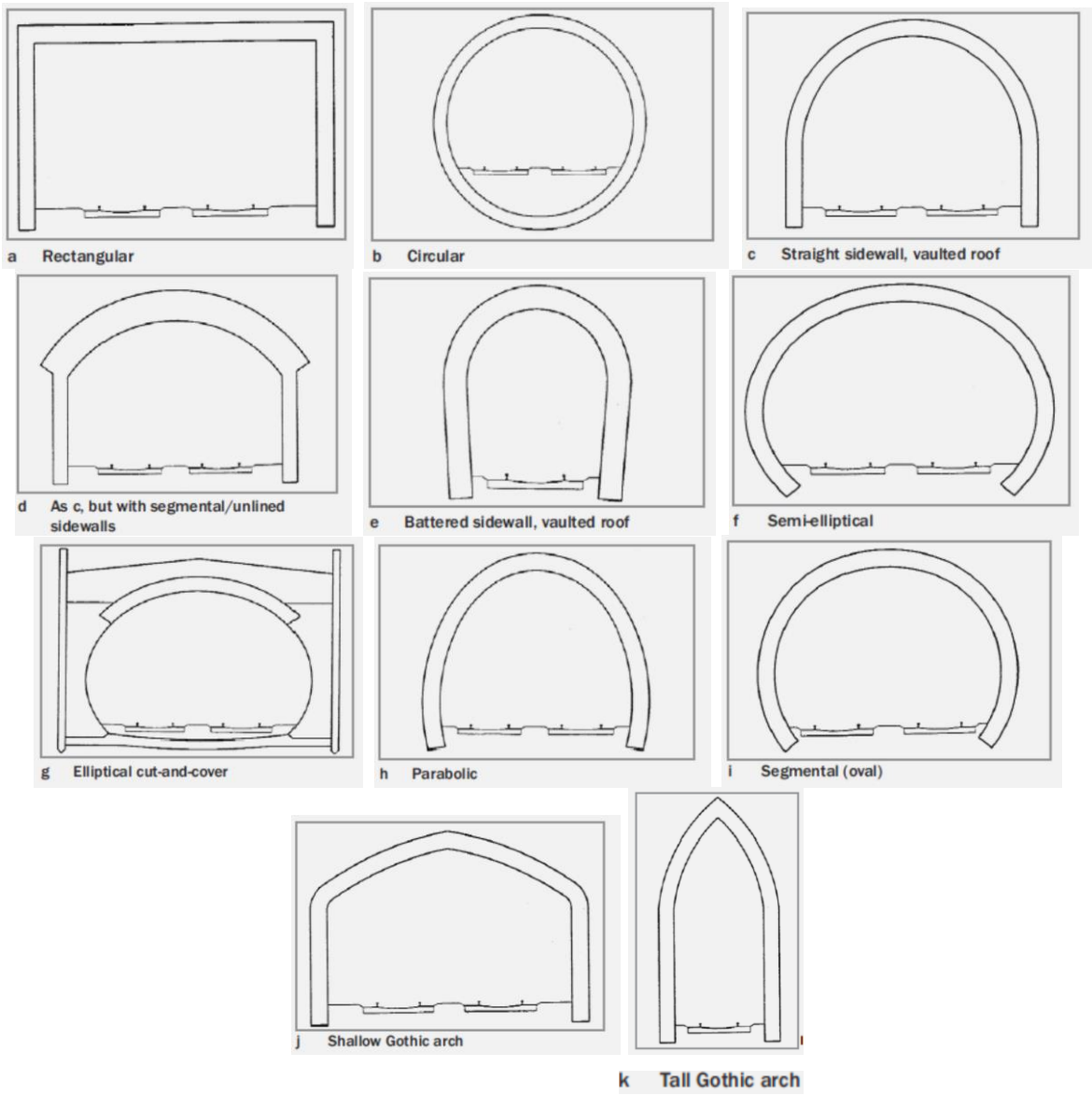


Figure 6-1. Typical UK tunnel cross sections (Railtrack 1996)

The material used to line tunnels can be masonry (either stone or brick), insitu concrete, pre cast concrete segments or metallic (cast iron or steel) segments and each presents its own challenges when life extension is being considered.

As most railway tunnels date back to the 19th century they were inevitably constructed using hand tunnelling techniques, which usually required the construction of vertical access shafts from ground level. In some cases these shafts were left open to provide ventilation but others were backfilled and are referred to as buried or hidden shafts whose location is often unknown. Construction of shafts are discussed in ML-D1.3 (2014)

As part of the process of determining the need to undertake life extension of a tunnel it is sometimes necessary to undertake a structural assessment. This will follow similar lines to that adopted for bridges constructed of the same basic engineering material and can include the advanced techniques described in chapter 4 of this guideline.

6.2 Life extension of unlined tunnels

The deterioration of unlined tunnels is usually limited to failures of the rock through which the tunnel is driven, often caused by ground water permeation or the effect of frost. This can vary from the regular loss of small pieces from friable rock to the dislodgement of large blocks of stone which could be large enough to derail a train as shown in Figure 6.2. The potential for the latter type of failure is largely dependent on the orientation of the bedding planes within the rock mass.

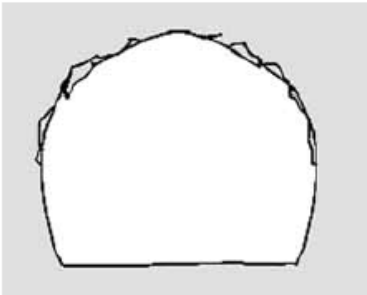
Life extension measures are largely traditional and can consist of (based on CIRIA C671);

- Scaling of loose material or the removal of loose blocks
- Provision of protection shelters
- Localised rock bolting
- Rock bolting and rockfall protection mesh
- Localised application of sprayed concrete to support weak areas of rock
- Installation of supporting ribs
- Installation of full lining, usually using sprayed concrete

1 Loose surface material

Loose rock on tunnel roof and walls is typically the result of poor blasting control during construction, weathering or washout of supporting material. Falling material is normally limited to relatively small debris but still pose a hazard to tunnel users and equipment.

Loose material should be identified and made safe, normally by removal or in some cases addition of a secondary lining.



2 Potentially unstable blocks/wedges

Rock blocks are bounded by discontinuities, the spacing and orientation of which allow kinematically inadmissible blocks. There is a risk of unstable rock falling onto or into the path of traffic and/or damaging tunnel equipment.

Rock stability should be checked (eg by scanline survey of joint sets) and the need for stabilising measures (eg rock bolting, application of mesh, sprayed concrete) assessed.

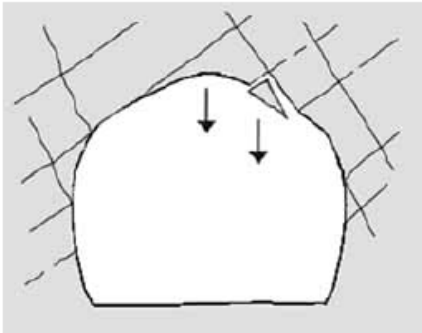


Figure 6-2. Defects in unlined tunnels (CIRIA C671)

6.3 Life extension of lined tunnels

6.3.1 Masonry

The deterioration of masonry in tunnel linings is very similar to the deterioration of masonry in arch bridges and can be due to:-

- Freeze thaw action
- Salt attack
- Sulfate attack
- Leaching of mortar
- Biological attack
- Use of unsympathetic repair material
- Thermal effects
- Saturation
- Ground movement
- Fatigue

These deterioration mechanisms usually manifest themselves as spalling of the masonry lining, cracking of the masonry lining or loose masonry caused by loss of mortar.

Traditional repair/life extension techniques usually consist of:-

- Patch repairs (see Figure 6.3)
- Crack repairs
- Ring separation repair
- Grouting
- Underpinning

- Invert repair

Figure 6-3. Traditional method of relining/replacing damaged brickwork using temporary metallic sweeps and wooden lagging to support new brickwork.

Novel products are now available on the market and are briefly described below.

- Electro osmosis can be used to firstly dry out wet patches in a tunnel wall and then, by reversing the polarity of the system, to draw grout into voids to both structurally strengthen masonry and inhibit further water ingress. When using this technique it is not a good idea to try to fully waterproof a tunnel, since that can lead to the build-up of highly dangerous hydrostatic pressure behind the lining. Instead water should be directed to suitable outlets, such as low level weep pipes, where the water can be successfully managed. A description of the method can be found at http://www.structural.net/article/electro-chemical-dewatering-system
- New support systems have been developed, which can be left insitu permanently. One example is shown in case study 2 in Appendix C in ML-D1.3 (2014)

6.3.2 Insitu concrete linings

Insitu concrete linings usually suffer from similar deterioration mechanisms as other forms of concrete structure (principally carbonation or chloride induced corrosion) and the repair methods also generally follow traditional lines. Advice on how to ensure quality repairs can be found in numerous publications and research project outputs; one example is the Rehabcon manual available at http://www.cbi.se/objfiles/1/MANUALmaindocum_1522366004.pdf

More recently the use of both steel and carbon fibre to strengthen tunnel linings has started to become accepted, particularly in Japan. Details of one steel system can be found at http://www.nssmc.com/en/tech/report/nsc/pdf/n9209.pdf and details of carbon fibre strengthening can be found at http://www.wtec.org/loyola/compce/04_07.htm).

6.3.3 Concrete segments

Tunnels lined with concrete segments can deteriorate in a number of ways, which can indicate different root causes as shown in Table 6.1 (after CIRIA C671), see also Figure 6.4.

Table 6-1. Deterioration of concrete tunnel segments

Deterioration noticed	Possible cause
Cracking within segment parallel to cross joints	Concrete shrinkage
Spalling of concrete within segment	Casting defects, impact damage, rebar corrosion, excessive loading
Spalling to edges of segment	Out of plane construction
Diagonal cracking within segment	Incipient compression failure
Lipping of joints between segments	Construction defect
Circumferential crack within segment	Overloading leading to possible bursting failure

Figure 6-4. Edge spalling of a concrete tunnel segment

Where excessive loading is suspected a detailed engineering assessment will be needed before deciding on the correct remedial action. For deterioration that is not structurally significant conventional concrete repair techniques are usually sufficient, but in more serious cases techniques such as those illustrated in Figure 6.5 will need to be considered.

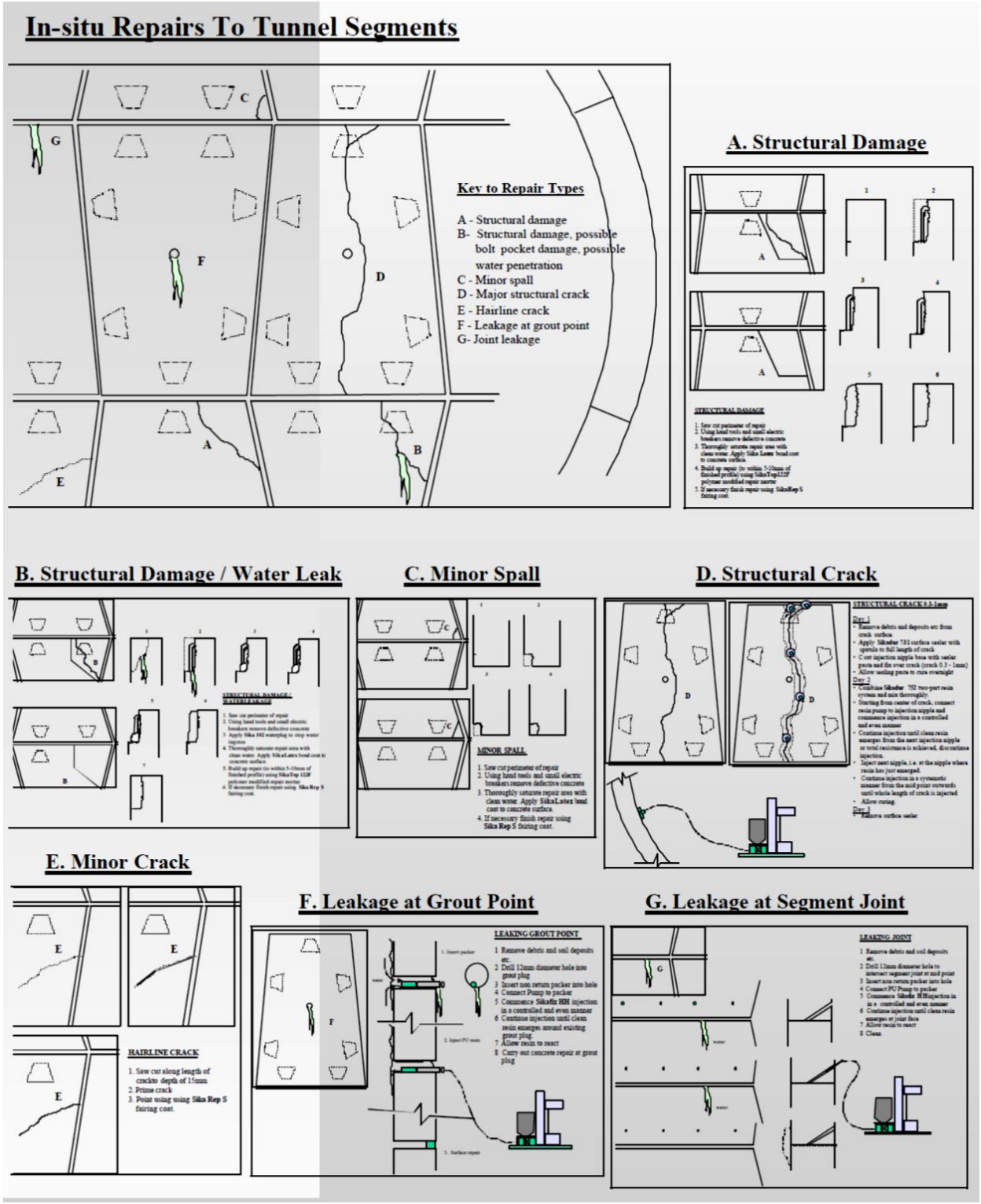


Figure 6-5. Repair techniques for deteriorated concrete segments

http://www.hammersmithgroup.com/datasheet/01/ref-10.pdf

In extreme cases the replacement of the lining will be necessary. This can be undertaken either by the installation of replacement segments or by the use of sprayed concrete and,

although expensive, can be carried out in very short lengths during overnight closures of the tunnel.

6.3.4 Metallic segments

Metallic segments can be manufactured out of either steel or cast iron and are usually only to be found in circular tunnels. The most common defects are cracking (principally at bolt holes) and corrosion. Steel linings can be repaired by welding or bolting new steel into position but cracked cast iron should be repaired using stitching. Corrosion principally affects steel linings and should be treated using conventional surface coating techniques.

There is at least one example of the replacement of cast iron segments with cast stainless steel segments in an area where sulphuric acid was present in the ground water (reported in Appendix A1.9 of CIRIA C671). The solution has been designed for a 400 year life and, whilst expensive, was deemed to be only practical solution in this particular case.

6.4 Life extension of tunnel shafts

Shafts in tunnels can be found in a number of different configurations and some guidance is given in ML-D1.3 (2014

Three case studies are presented in Appendix C in ML-D1.3 (2014).

7. Track and Earthwork

7.1 Summary

Overviews of practice and research are presented in e.g. Lothar (2007), IHHA (2009) and Innotrack (2010). Track-wheel interaction is treated in Lewis and Olofsson (2009). General information on the history of tracks is given in Pike (2010). Many experiences as a railway engineer in the U.S. is shared by Selig (2014). Recent trends are given in Veit and Marschnig, (2011, 2012).

A typical double track and its parts are illustrated in *Figures 7.1* and *7.2* Below new and improved methods are summarized in a table. The different methods are then presented in some more detail in the following subsections.

Figure 7-1. Double track and embankment in Torp, Sweden, Innotrack (2010)

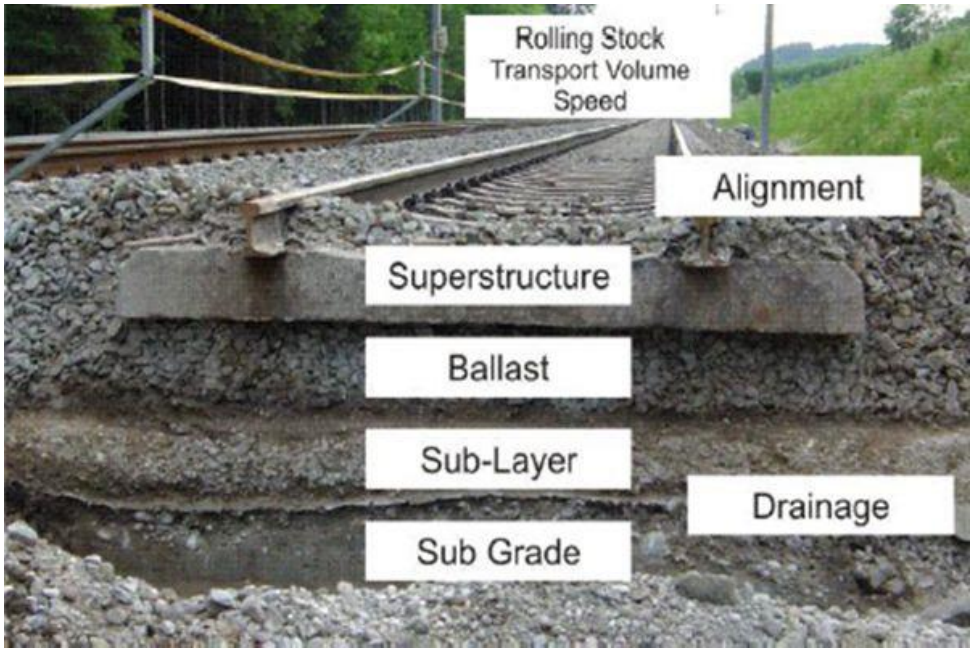


Figure 7-2. Track definitions, ML-D5.6 (2014)

Work on track systems is or has been going on in the European projects Automain (2014) during 2011-2014, Interail (2012) during 2009-2012, Smartrail (2014) during 2011-2014 and Sustrail (2014**)** during 2011-2015**,** In Sustrail D1.2 (2012) examples are given on track conditions for three routes in Bulgaria, Spain and the U.K.

It can be seen from the Questionnaire in appendix A in ML-D.1 (2013) that not many Infrastructure Managers yet use Life Cycle Cost Assessment in the planning of maintenance and repair of their rail infrastructure. There is a lack of data and methods and here the MAINLINE project with its Life Cycle Assessment Tool (LCAT) may give assistance, see ML-D5.7 (2014).

Table 7-1. Summary of technologies to improve track – pros and cons

New Technology	Pros	Cons
Better inspection and assessment methods	Improves safety and economy	Implementation may need some additional work
Grinding of rail with optimized procedures	Improves safety and economy	Takes time and and may cost money in the short run
Improved Switches and Crossings	Fewer failures	May initially cost more
New strategies for tamping and ballast cleaning	Improves ballast properties	May be more expensive in the short run
Improved sleepers to delay degradation	Last longer	Quality control is necessary

7.2 Better Inspection and Assessment Methods

Inability to detect rail cracks at an early stage of growth hinders the planning of mitigating actions such as grinding. It may also entail that cracks are allowed to grow too long before removal, which leads to higher grinding costs and more operational disturbances, and also to a shorter rail life. In severe cases this may even be a safety issue.

Inspection methods and equipment to detect rail cracks were studied in Innotrack (2010), see Ekberg and Paulsson (2010). The work continued in Automain (2014), Interail (2012) and PM'n'IDEA (2012). In Automain 2.3 (2014) five inspection technologies were developed:

- infrastructure mounted video cameras
- high speed inspection of switches using lasers
- SIM (Switch Inspection Measurement) wagon
- in-service track geometry recoding
- instrumented washers and alternative approaches to securing track components

In addition, three higher level enabling technologies were also considered:
- the development of a suitable Man Machine Interface
- modular components to facilitate rapid track maintenance, ideally incorporating facilities to support automatic inspection

- the potential for self-powered wireless sensors to be used to monitor or measure various aspects of the infrastructure

The report Automain 2.3 (2014) also includes a concept design for a self-inspecting switch and supporting systems

7.3 Grinding of rail with optimized procedures

Grinding is used to increase rail life and reduce cost, see Zarembski (2005). Grinding costs are today relatively high. Two reasons for this are poor logistics planning and lack of network grinding strategies. Innotrack (2010) has delivered a guideline and a Technical Recommendation on optimized grinding procedures. This document includes not only technical specifications (e.g. profile tolerances), but also logistical and strategic considerations.

High speed grinding (HSG) is a relatively new proprietary method based on the principle of circumferential grinding. Cylindrical grinding stones are pulled over the rail at an angle, inducing rotation as well as an axial grinding motion. The grinding stones are mounted on grinding units hauled by a carrier vehicle. In this way the speed can be increased to 60-80 km/h compared with traditional grinding with a much lower speed. Fast ultrasonic testing (search for head checks) can support fast grinding procedures.

In Automain 4.2 (2013) the following conclusions are drawn regarding optimization of grinding:
• Long enough maintenance windows are necessary to minimise track possession time needed for grinding.
• The use of improved conventional grinding with 64 stones shows that reduction in track possession time in the order of 50 % is possible. This reduction depends on the layout of the track.
• HSG and twin HSG present good opportunity for the reduction of track possession time, in comparison with conventional grinding over 67% reduction in track possession time is possible.
• The grinding cost is in the order of 5 -10 % of the total LCC for the rail and the most significant cost element is the cost of track replacement. This makes decision on time to replace track crucial from LCC perspective.
• An improved conventional grinding machine will have about the same order of cost as the High Speed Grinder, but will most probably give earlier replacement of rail than the High Speed Grinder so the LCC-cost is slightly higher.

7.4 Improved switches and crossings

Switches & crossings (S&C), also named turnouts, are discontinuities in the track systems. They impose dynamic loads on track and rolling stock and are prone to mechanical failures. Two types of improvements may be possible: (1) more robust switches with new types of components less susceptible to failure and (2) improved maintenance procedures.

Through numerical simulations calibrated from in-field measurements recently several measures to optimize the mechanical characteristics of S&C have been proposed, by Nissen (2008) and Innotrack (2010). Work is now going on in Automain (2014) and Sustrail (2014).

For maintenance of switches and crossings Automain 4.2 (2013) found that
• Value stream mapping together with simulation makes a time saving of 50 % in crossing replacement possible. However, it requires two welding teams compared to one in current practice.
• In the study on optimal maintenance window in-between regular departures, a maintenance work of 120 minutes was simulated with regular time tables at different frequencies. Results indicates that 40 minutes train frequency could be considered as an optimal window size regarding train service and maintenance cost. It gives 35 % saving compared to a train frequency of 20 minutes.

These models and simulations may be implemented after suitable modifications as relevant to the IMs for achieving an anticipated 40% maintenance possession time reduction.

7.5 New strategies for ballast tamping and cleaning

Over time, ballast settle and the track may be in need of an alignment. This can be done with a ballast tamper or tamping machine which is used to pack (or tamp) the ballast under the railway sleepers. With modern machines the track as a whole can be levelled, aligned and tamped, in order to achieve a smoother ride for passengers and freight and to reduce the mechanical strain applied to the rails by passing trains.

The ballast also becomes worn with time, and loses its angularity, becoming rounded. This hinders the interlocking or tessellation (from Latin tessera = square) of pieces of ballast with one another, and thus reduces its effectiveness. Fine pieces of granite, like sand, are also created by attrition, known simply as "fines". Combined with water in the ballast, these fines stick together, making the ballast like a lump of concrete. This hinders both track drainage and the flexibility of the ballast to constrain the track as it moves under traffic.

Ballast cleaning removes the fine material in the worn ballast, screens it and replaces the "dirty" worn ballast with fresh ballast. The advantage of ballast cleaning is that it can be done by an on-track machine without removing the rail and sleepers, and it is therefore cheaper than a total excavation. It can be carried out with e.g. a ballast cleaner which is a machine that specializes in cleaning the railway track ballast (gravel, blue stone or other aggregate) of impurities.

In total, over the whole life of track, tamping is the most frequent measure in maintenance. Rail exchange or a ballast cleaning is far more costly than tamping (factor up to 10 times more expensive). Together tamping and cleaning of ballast are usually the processes that are most costly in maintaining a railway track. The strategy for how often these maintenance procedures are carried out have a vast influence on the overall cost and quality of the track, Veit and Marschnig (2012).

In Automain 2.4 (2013) a case study showed that:
• Adequate maintenance windows lead to reduced track possession time that will be required on the long run. Around 5 hours would be optimum since the benefit of further increasing the maintenance is very small.
• Optimum possession length is required to reduce the impact of necessary non-value added tasks.
• The behaviour of the track becomes unreliable if the tamping cycle becomes too large or in the absence of one owing to increased number of spot failure with high variation in track possession time.
• Improvement of tamping speed gives the highest reduction in track possession time. 10% improvement in the tamping activities gives 11% reduction in the track possession time while 40% improvement gives about 35% reduction.

• The number of isolated geometry failure over time follows a power law process. Following this, an optimum strategy from track possession point of view is to have a tamping interval of 6 years.

Stone blowing is a relatively new process of track geometry adjustment that involves adding crushed rock to the surface of ballast under the lifted tie to shim the track. This is an alternative to tamping in which the ballast particles are rearranged to fill the void under the lifted tie. To begin the stone blowing process, the existing geometry of the track is measured. Then, the vertical adjustment at each tie required to achieve the desired geometry is calculated. Next, the volume of stone to be blown beneath each tie to achieve this adjustment is determined. The stone is then placed by lifting the ties, inserting blowing tubes, and blowing the stone under the tie. Train traffic will then seat the ties as the blowing stone particles settle in. Stones used for blowing are to be selected to optimize their use for shimming., Selig (2014)

7.6 Improved sleepers to delay degradation

The design of prestressed sleepers, has been studied by Gustavsson (2002) and Charmec (2012). In a recent design guide, Bolmsvik et al (2011), conclude that the conditions at the wheel–rail interface and of the sleeper support have significant influences on the bending moments generated in the sleeper. Hence, regular and controlled maintenance of rolling stock, rails and ballast bed, are prerequisites to optimize the pre-stressed concrete sleeper.

An innovative new sleeper design is to enlarge the contact area between the sleeper and the ballast by applying a polyurethane layer on the bottom of the sleeper, so called under sleeper pads, USP. This also gives an additional elasticity to the track, Schneider et al (2011), Veit and Marschnig (2012) and Charmec (2012).

When prestressed concrete sleepers are produced it is important: (1) that cement of the correct quantity and quality is used and (2) that the curing process is properly controlled so that the temperature does not get too high and hence lead to delayed ettringite formation (DEF) after some years. This has happened in several countries time after another, see Tepponen and Eriksson (1987) and Famy and Taylor (2001). Recommendations have been given in CBI (2000), Scrivener and Skalny (2005) and EN 13230 (2009). Sleepers with limited partial cracking fulfil the load-carrying requirements. They are quite robust, and small cracks do not seem to influence the load carrying capacity or the fatigue resistance and it is first when the cracking is very severe that the load carrying capacity is reduced significantly, Thun et al (2008). However, this may finally be the case with delayed ettringite formation and then most of the damaged sleepers have to be replaced. So, it is important to keep an eye on the production so that proper cement and a good hardening process are used.

7.7 Earth Work

A general presentation of geotechnical problems and solutions regarding stabilization and scour protection in Earthwork is given in Fell et al (2005) and in Selig (2014). Generally it can be said that predicted climate changes may cause intensified periods of rain and drought in various places on the earth. This will demand that traditional designs using classic 1000 year values need to be revised. Some special areas are discussed below, see Table 7.2. Other areas are treated in Smartrail (2014) as transition zones.

Table 7-2. Summary of technologies to improve earthwork – pros and cons

New Technology	Pros	Cons
Slope stability analysis for progressive failures	- More accurate predictions	- More complicated calculations
Lime-cement columns	- Stabilizes the ground	- Need special equipment to apply
Vertical drains	- Stabilizes the ground	- Need special equipment to apply

Sub grade (natural ground) - Degradation processes consist mainly of settlement due to creep, and erosion leading to lack of shearing strength and bearing capacity failure. A recent method to analyse the risk for progressive landslides in long natural slopes was presented by Bernander (2011). The problems with progressive landslides are especially important in countries with strain-softening clays as e.g. Scandinavia and Canada.

A fairly new method to stabilize the sub-grade is to use the so called lime–cement columns, see e.g. Larsson et al (2012).

Cuttings and Embankments - Degradation processes consist mainly of erosion and creep deformations leading to scour and loss of stability. A method to improve the embankment for the Ådal line in northern Sweden is presented in Müller (2010). The line is founded on sulphide clay and the stability was improved by using preloading in combination with prefabricated vertical drains. Additional information on vertical drains is given in e.g. Hansbo (1997).

Retaining/sea walls - Degradation processes consist mainly of erosion and creep deformations leading to scour and loss of stability.

8. Economic and environmental assessment

8.1 Economic assessment

Europe has a railway network of some 230,000 km with an asset value of more than 1500 billion € (Giga € or G€) and is spending – with large variations - less than 1% of it for yearly maintenance. A large proportion of the civil engineering structures and tracks are old; of the 500,000 bridges, 35 % are over 100 years old and earthworks and tunnels are often older. Nonetheless they can, with the help of the results from MAINLINE, remain in service for longer periods, improving the ability of the railways to deliver increased mobility across Europe and play an increasingly important role in the development of integrated, safer, “greener” and “smarter” pan- European transport systems.

A modest 10 year increase in the service life of 2% of the bridges due to the methods presented here would give that the replacement of 10,000 bridges could be postponed for 10 years with notional cost savings calculated below.

- The average construction cost (K) of a new railway bridges is about 1M €
- With a low interest rate (p) of 2% the present value of the cost for rebuilding a bridge in 10 years will be K/(1+p)10 = 0.820 K
- Compared to rebuilding the bridge now the saving will then be K – 0.820 K = 0.180 K.
- Not replacing 1,000 bridges per year gives a saving of 1,000 x 0.180 x 1 M€ = 180 M€.
    -

Similar savings will be available for other kinds of infrastructure; for instance significant performance enhancements for Switches and Crossings (S&C) have been developed. There is potential to enhance replacement methods of S&C by reducing replacement time, ensuring good track alignment and reducing varying track stiffness. This has been one of the principal focus areas for MAINLINE and the potential benefit is outlined below.

- There are approximately 150,000 switches in mainline track in Europe.
- Switches are changed about every 20 to 30 years
- The cost to change a switch is of the order of 0.15 M€
- This gives a yearly cost of some (150,000/30) x 0.15 M€ = 750 M€.
- If the quality of the switches could be improved so that they would last some 25 % longer this would lower replacement costs and thus save 750 – (150,000/37.5) x 0.15 M€ = 150 M€ per year.

So, just from savings on bridges and switches and crossings we may reduce costs by more than 300 M€ per year. Additional savings will arise from the MAINLINE results for plain line track and soil cuttings, which have not been quantified here.

8.2 Environmental Assessment

Traditionally the main environmental consideration associated with railway operation has been related to reducing noise and vibration, but with the main environmental focus now

turning to climate change and the associated carbon agenda, new considerations are becoming important. In common with many other parts of the built environment, the carbon impact of railway infrastructure is dominated by usage rather than initial construction or ongoing maintenance. However if the railway industry is to play its part in meeting the carbon reduction targets set within Europe, then the carbon impact of infrastructure maintenance and renewal activities will have to decrease.

Within the MAINLINE project a tool has been developed that enables Infrastructure Managers to assess the environmental (carbon) impact of various maintenance or renewal interventions under consideration and hence have the opportunity to select the one with the least impact. Unfortunately the current state of knowledge about the carbon impact of typical interventions is limited and largely confined to academia, which means that it is virtually impossible to quantify the benefits from the project; however the example below will give an indication of the kind of benefits that could be realized.

- The emission of carbon dioxide from the building of a concrete bridge containing 160 m^3 of concrete can be calculated in the following way:
- 1 m^3 of concrete weighs approximately 2.3 tonnes.
- Concrete contains about 400 kg of cement per tonne
- Cement production creates approximately 700 kg CO_2/tonne
- This gives 160 x 2.3 x 0.4 x 0.7 ton = 103 tonnes CO_2.
- To this we add 150 kg reinforcement steel per m^3 of concrete.
- Steel production is responsible for some 1,2 kg CO_2/kg steel
- This gives 160 x 0.15 x 1.2 tonne = 29 tonnes CO_2.
- In total we will emit about 103 + 29 >> 130 tonnes CO_2 per new bridge.
  -

Research projects developing a new generation of lightweight, low energy, self-compacting concretes for structural applications have shown that it is possible to replace substantial quantities of cement with PFA (pulverized fuel ash) or GGBS (ground granulated blast furnace slag) without affecting structural performance, Pike et al (2009). This can save around 40% of the embedded energy in concrete, which would mean a reduction of 40 tonnes CO_2 from the concrete – equivalent to a 30% reduction in overall carbon footprint for such a bridge.

The potential for savings in a new build scenario can be demonstrated by reference to the Environment Product Declaration prepared for 190 km of a new single track railway with 90 bridges of a total length of 11 km and with 25 km of tunnels, Bothniabanan (2010). Per km the bridges were calculated to emit 8 050 ton CO_2 equivalents and use 22 GWh (80 TJ) during construction and 60 years of maintenance. The energy use per km of tunnel was of the same magnitude and the emission was about half of that of the bridges. Per bridge this gives in average an emission of 1020 ton CO_2 equivalents and a use of energy of 2,65 GWh (9,5 TJ). So, the savings in extending the life of existing structures instead of replacing them are large.

8.3 Open questions

To achieve sustainable transportation and low carbon emissions there is a need for

- Efficient utilization and maintenance of the rail infrastructure. Rail utilization will increase in the future if the carbon dioxide emissions from airborne traffic are subject to the same taxation as surface transportation.

- Increased use of strengthening to extend the life of existing structures. Full scale tests to failure of obsolete structures may give guidance on which strengthening methods that have the best function and sustainability

- Increased use of efficient assessment methods. Infrastructure Managers should be encouraged to learn to use and familiarize themselves with these methods. As an example a new method for estimating reinforcement corrosion degradation saved 3 M€ on two bridges in Stockholm

- Maintenance efficiency can be improved by better methods and data for RAMS (Reliability-Availability-Maintainability-Safety/Supportability)

- Better LCAT methods with more data and more options for different alternatives than the first important steps which are taken in the MAINLINE Project.

9. Conclusion

The most cost-efficient way to extend the life of elderly infrastructure is to maintain it properly and to know and understand how it functions.

Regarding bridges there is much to be gained using adequate assessment methods. The methods prescribed in our Eurocodes are often designed to give safe structures under all construction conditions. Regarding existing structures the codes are often conservative as there are fewer uncertainties regarding actual geometry and material properties than in not yet built structures. New probabilistic methods are useful and additional help may be gained from finite element models and studies of redundancy and robustness. Proof load testing, carried out in a careful way, may also give information that can be used to calibrate models. Full scale tests to failure of old bridges, which are to be taken out of service, is another possibility. They can give valuable results regarding the capacity of the kind of bridges that is tested.

Strengthening of structures is another important possibility to extend life. Some examples are given in chapter 5 and in Appendix A

Methods for tunnels are presented in chapter 6 and for track and earth work in chapter 7.

10. References

A more comprehensive list of references is given in ML-D1.3 (2014)

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Appendix A-H. Table of contents

Appendix A – Strengthening of concrete structures

A.1 Traditional concrete improvement methods - General

There exist several traditional methods for strengthening of concrete structures. For strengthening of railway bridges the following strengthening methods have been identified as traditional methods, and will be described in this appendix:

- Increased cross-section
- Post-tensioning and clamping
- Stitching
- Insertion of additional reinforcement
- Injection

In addition to these methods there are some alternatives that sometimes are referred to as strengthening methods. For many concrete bridges, definitely not all, the concrete compressive strength has improved over time, i.e. the capacity is sometimes better for an old structure compared to when it was newly built. This phenomenon is understood but cannot be significantly affected and not be used as a strengthening measure. Concrete bridges may, as most other structures, be improved by insertion of extra members. Insertion of extra members rather changes the structure than strengthen it and is therefore not considered to be a strengthening method. Further information is given in SB-STR (2007) and SB 6.2 (2007)..

A.2 Increased cross-section

Increased cross-section may improve the capacity of a concrete structure itself but are commonly used together by installation of additional reinforcement. In such cases, the added concrete transfers the forces between the new reinforcement and the concrete structure. The concrete layer also serves as protection for the reinforcement. The concrete can be placed on the member with formwork or by shotcrete. The reinforcement can be in form of bars or nets of steel, stainless steel or fiber reinforced polymers. The added concrete can also be reinforced by fibers that serve as reinforcement alone or together with traditional reinforcement. The first mode of action is to remove poor or chlorine affected concrete and if necessary take care of deteriorated reinforcement. After this step, the additional reinforcement can be placed. During installation of additional reinforcement it might be necessary to locally weaken the original structure as shown in Figure where stirrups have been anchored in the flange of a T-girder. Finally, the new cross-section and the new concrete cover are built up.

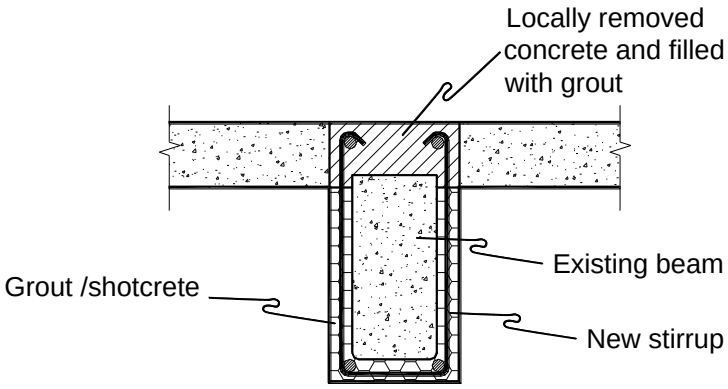


Figure A.1: Cross-section of a strengthened beam

In some cases the strengthening system is designed to carry the loads itself and the old concrete will serve more as formwork or extra safety against failure. This cannot be done without

thoroughly analysis because the ductility can be dramatically changed. This strengthening method does have a number of drawbacks. By adding material to the structure the dead weight of the structure is increased with change of natural frequency as a consequence. There are uncertainties about the bond between the old concrete and the new concrete. With temperatures due to hydration of cement and shrinkage in combination with restraint from the old concrete, cracks can arise in the new concrete. It has been found that loads during the work can have negative effects on the strengthening effectiveness. The method is rather costly and involves large amount of material transportation. The advantages with the method are that the materials used are well known and there exists a widespread knowledge how to do this kind of work.

A.3 Post-tensioning and Clamping

When concrete structures have a reserve compressive capacity, post-tensioning may be a suitable candidate for strengthening. For the tendons to post-tension, both steel and carbon fiber composites can be used. Post-tensioning with longitudinal strands, i.e. flexural strengthening, also increases the shear bearing capacity of a beam by changing the stress and strain situations. Post-tensioning may also be applied in the transverse direction to directly improve the shear bearing capacity. This contribution is considered in some codes by adding a term to the total shear capacity. Post-tensioning has been used quite extensively for strengthening of structures in flexure, especially for structures post tensioned from the beginning. When adding post-tensioning cables to a structure it is often necessary to make arrangements for anchoring the cables. These arrangements will alter the structure and rather high secondary moments are introduced and needs to be analyzed. One important consideration is to prevent corrosion on the new strands. Also, to ensure quality and to prevent damaging of the structure the work needs to be done by skilled workers. The advantage of this method is that the post tensioning can be a fast procedure with only small interruptions in the traffic and that there exists methods for quality assurance. The prestressed tendons can be placed externally or internally in drilled holes, see Figure, which shows an example of shear strengthening.

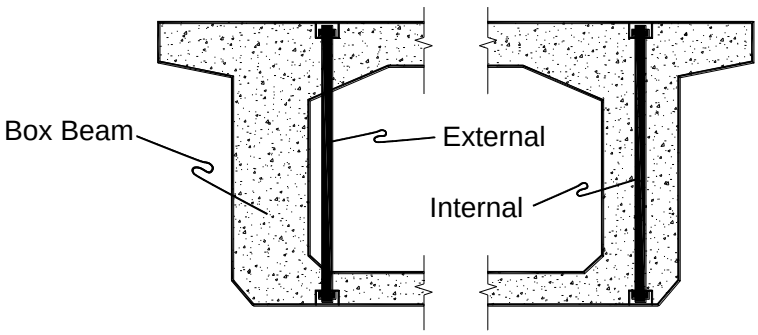


Figure A.2: Inserted and prestressed reinforcement

For all techniques of post tensioning the tension forces must be transferred into the concrete. This can be done over the whole length, at the end of the strands, or a combination thereof. The high transferring forces must be considered and securely anchored. Sometimes it is necessary to cast anchorage blocks and diaphragms for the tendons. Those blocks can be quite big and sometimes problematic to make because of limited space. When the work with the anchorage arrangement is done the tensioning work starts. The strands are prestressed with hydraulic jacks and then anchored in place. When the anchorage is effective the jacks are released and the strengthening is complete.

An alternative of post tensioning in transverse direction is called clamping. In this case bolts on the beam sides anchored with nuts and plates on top and bottom of the member are used to form a shear reinforcement that clamps the beam, see Figure. The method is mostly used without pretensioning of a hydraulic jack. The bolts will instead be prestressed from tightening of the nuts.

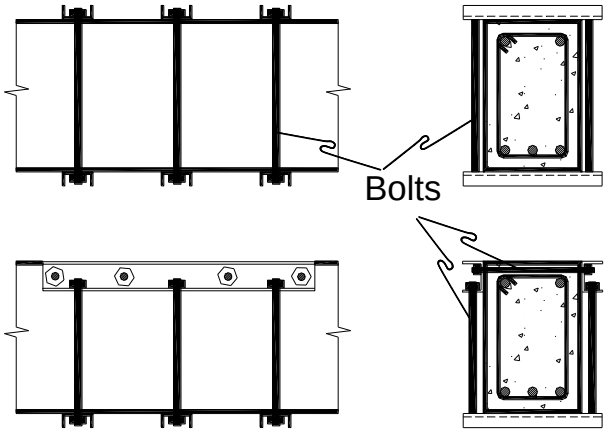


Figure A.3: Clamping around the beam and on three sides.

A.4 Stitching

A crack in concrete can be sewed together to be able to transfer tension forces, see Figure. The stitches are made of hooked steel bars placed preferably perpendicular to the crack. The stitches are normally anchored rather close to the crack. This means that the tensile forces will be moved to the end of the stitch and if all stitches are anchored along one line it can give a new crack. The stitches should instead be placed at different angles and in different lengths to provide anchorage over a larger area. The method improves and stiffen the structure locally by strengthen cracks that has occurred due to a temporary overload or a local weakness.

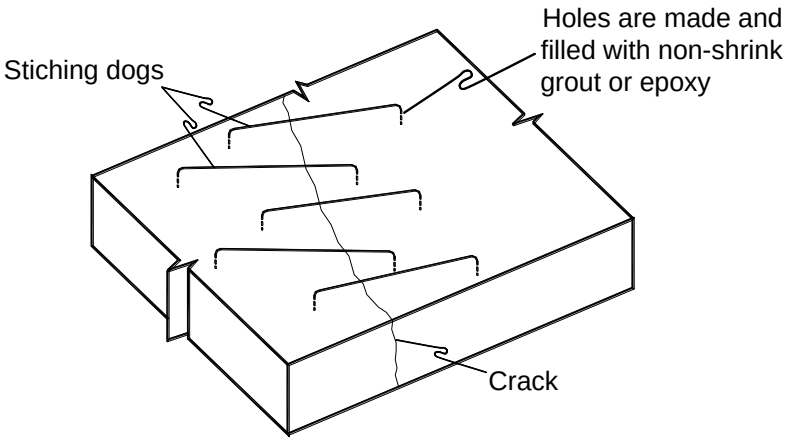


Figure A.4: Stitching of a crack

A.5 Insertion of additional reinforcement

Another way to get additional reinforcement is to place bars inside of the concrete. In this case holes are drilled and rods are inserted. The added reinforcement can either be made of steel or fiber reinforced polymers. The rods are bonded to the concrete with an adhesive. The effectiveness of the method is strongly dependent on good adhesion between the rods and the concrete. The technique can also involve pretensioning of the rods. The method can

be used for both flexural and shear strengthening. An advantage for the method is that the concrete protects the reinforcement.

A disadvantage is that the method can weaken the structure when the holes are drilled especially if the drilling damages the existing steel reinforcement. Another problem may be the difficulties of ensuring that the bonding process is correctly undertaken. An example of shear strengthening is shown in figure.

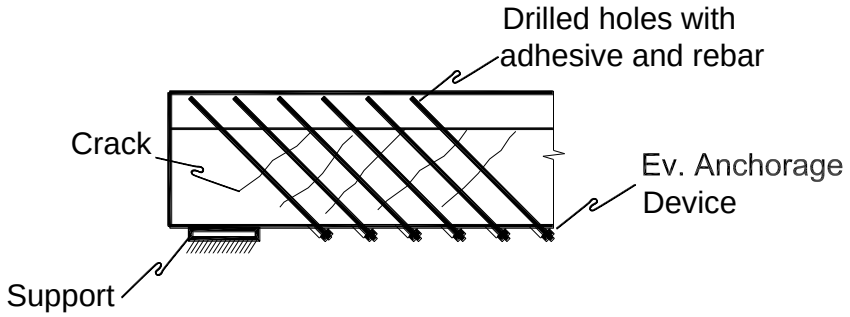
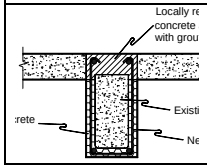
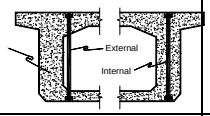
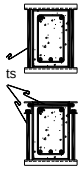
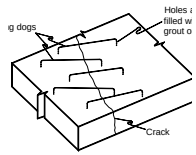
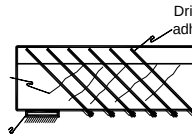


Figure A.5: Insertion if extra reinforcement

A.6 Injection

A crack might need to be taken care of before strengthening. Injection of a crack from temporary overload can also be seen as a strengthening method. Injection can, depending on the width and depth of the crack be done with epoxy resin or cement grout. Cracks as narrow as 0.05 mm can be bonded by epoxy injection. If it is only a few localized cracks then the cracks can be injected by placing nipples along each crack. The resin will then be pumped into the crack via the nipples. If the cracks are formed in a pattern or consist of a large number of cracks in a limited area then it can be more effective to use a vacuum process. Cracks over 1 mm can be injected with cement grout. With increasing crack size grout becomes more economical than epoxy. Resin injection can restore a structure to its original design capability and prevent further degradation of the structure.

Table A.1. Overview of existing strengthening methods for concrete bridges

	Method	Principle	Traffic Disturbance	Stiffness	Bending	Shear	Tor-sion	Cost estimate	Experience with existing railways
	Increased cross-section	Increases height, width, internal lever arm, and moment of inertia of a cross-section	Yes	Yes	Yes	Yes	Yes	High cost	Some
	Post-tensioning	Reduces tensile strains and stresses	Minor	Yes	Yes	Yes	Yes	High cost	Some
	Clamping	Adds stirrups externally	No	Yes	Yes	Yes	Yes	Medium cost	Some
	Stitching	Overbridging cracks.	No	No	No	Yes	Yes	Low cost	Poor
	Insertion of rein-forcement	Adds reinforcement inside structures	Yes	Yes	Yes	Yes	Yes	Low cost	Some
	Injection		No	Yes	No	No	No	Low cost	Some

Appendix B – Strengthening of metallic structures

The traditional strengthening of metallic structures includes three topics that must be treated separately. Static load capacity, stiffness and fatigue strength. These main categories have a quite large variety of strengthening methods attached to them today, but not all methods are good for all purposes. Strengthening methods that are very good for increasing the static load capacity and stiffness of a structural member is often not permissible if the structure needs to be redesigned for fatigue. The other way around, most of the fatigue strengthening methods involves altering the original structure by removing material e.g. stop holes. This will weaken the net cross section the member and decrease the load capacity.

Many of the static strengthening methods will require disturbance of the traffic flow, because the load needs to be as low as possible on the structural member for the strengthening effect to be good. Ballast plates, rails, sleepers might need to be removed to perform the proper strengthening. This means that new methods which deal with easy material handling, installing equipment and quality control is needed in this area.

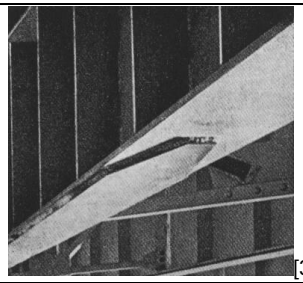
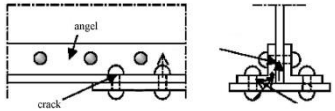
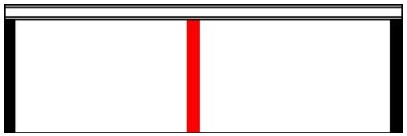
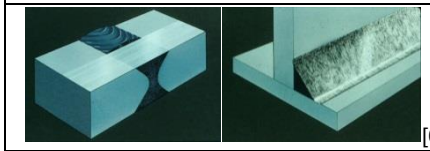
In the area of fatigue strengthening, the procedures are often limited to a small but critical area of the bridge, and are less likely to disturb traffic. However, if the traditional fatigue repair becomes very far-reaching, a disturbance of railway traffic is likely to occur. The fatigue strengthening methods can be described as ones removing the crack by mechanical methods or temperature methods and methods involving introduction of beneficial compressive stress zones near the damage or cold working of the metal.

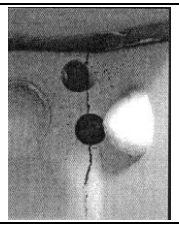
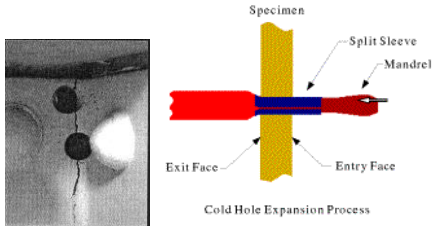
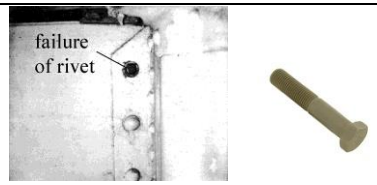
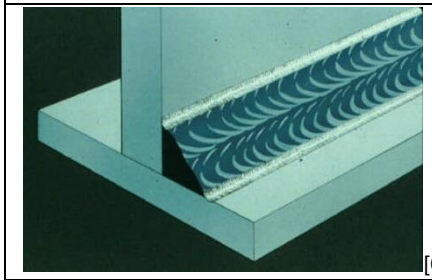
The strengthening methods shown in table B.1 are compiled to give an overview of a wide variety of old and new methods. Some methods are not valid for bridges made by old metals, since welding cannot be allowed here, but then other methods are shown. Certain methods are only good for some purposes or structural members. Some methods may be best on butt welds and some structures might be sensitive to drilling of holes. For such a detailed description, SB-STR (2007) and SB 6.2(2007) give an overview.

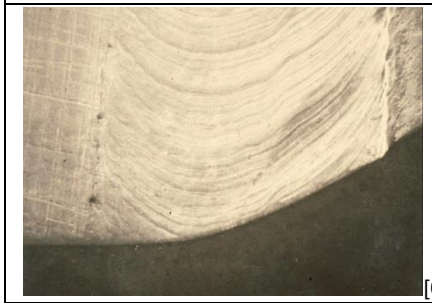
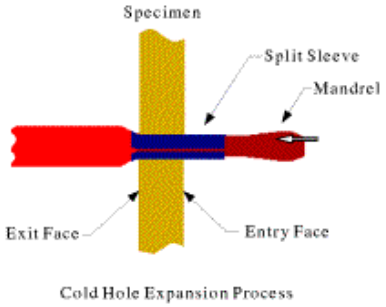
Before a strengthening is carried out, a sensitivity analysis on the traffic disturbance, evaluating the most relevant techniques must be performed. Since a lot for the strengthening methods are dependent on which details they affect, one might benefit of choosing an expansive rehabilitation form, but with no traffic disturbance.

Monitoring and quality control depends entirely on the selected method.

Table B.1. Overview of existing strengthening methods of metallic structures

	Method	Principle	Traffic Disturbance	Stiffness	Bending	Shear	Fatigue	Cost esti-mate	Experience with existing railways
 [3]	Cover plates Welded steel plates	Cover plates in steel are welded to flanges or web	Yes(depends on structure)	X	X	X		xx	Some
	Cover plates Bolt-on steel plates	Cover plates in steel are bolted to flanges	Yes	X	X			xxx	Some
 NY	Addition of new mem-bers	New members are added to the construction to insure carrying capacity for a part of the structure, e-g- span shortening. (Outcome of the method is dependable on the exchanged member). A column is shown here.	Yes	X (overall structure)	X	X	X	xxx	?
 [4]	Confine-ment with reinforced concrete	Jack up of structure and casting of reinforced con-crete around the structural member. Mostly applicable for columns.	Yes	X	X	X	X	xx	None?
	Air arch gouge and fill	Weld is cleaned by melting the metal and blowing it away. New weld material is added. Possibility for intro-ducing new defects with the new weld.	Yes(depends on placement of weld)				X	xxx	Some
 [6]	Disc grind-ing	Weld is smoothed to reduce and remove stress raisers. Bad grinding may introduce new defects.	No				X	x	Some

	Method	Principle	Traffic Disturbance	Stiffness	Bending	Shear	Fatigue	Cost esti-mate	Experience with existing railways
	Hole drilling	Stops cracks by decreasing stress intensity factor. Method is almost only applicable at large visible cracks. Many cracks will relatively fast reinitiate crack growth.	No	-	-	-	X	x	Extensive
 [5]	Hole drilling and cold working of bearing	As above, but the hole is cold worked with a tapered mandrel. Introduces com-pressive stresses near the bearing and thereby de-creases the stress intensity factor and risk of crack initiation	No	-	-	-	X	x	Yes
	Exchange of fastener to HS-prestressed bolt	Removes high bearing stresses and introduces triaxial stress state, benefi-cial for reducing risk of crack initiation	No	(X) in the connec-tion	(X)	(X)	X	X	Yes
 [6]	Burr grind-ing and polishing	Burr grinding grinds away / reshapes the critical part of most welds, the weld toe. Quality control is well de-veloped.	No				X	xx	Yes

	Method	Principle	Traffic Disturbance	Stiffness	Bending	Shear	Fatigue	Cost esti-mate	Experience with existing railways
 [6]	Laser/TIG remelting	Remelting the weld to eliminate defects and gaps within the weld. Best to do at a workshop with controlled conditions. Usable for up to 6mm defects. Picture is laser remelt	Yes(depends on location)				X	xxx	No?
 [6]	Peening methods. Impact treatment	Cold working of the weld toes by using rods or air blasting surface with steel and glass pellets. Picture shows performance of peening at weld toe and shot peening surface.	No				X	X(x)	No?
 [5]	Cold working holes	A tapered mandrel is pushed through the hole. Introduces compressive stresses near the bearing and thereby decreases the stress intensity factor and risk of crack initiation	No				X	x	Yes

Appendix C - Strengthening of masonry arch structures

It is vital from the outset that a holistic approach is taken to the assessment and repair/strengthening of masonry arch bridges. To change the nature and/or stiffness of one element of the bridge may only result in overstressing of another.

It is essential that the cause of the deterioration is understood and the effects of any strengthening or repair techniques are considered before commencing any work on the bridge. At all times it is important to consider that masonry arch bridges derive their strength and tolerance to movement from their ability to articulate – it is their particulate nature that gives them their unique structural characteristics. If this is lost then they behave in a different way that should be taken into account when considering their strength and residual life.

If remedial or strengthening works are selected without due consideration and understanding of their short and long-term effects, they may result in more harm than good and can address one failure mode only to allow, or even cause, another. Particular care is required when work is necessary to one span of a multi-span bridge.

Wherever possible, repairs must be sympathetic to the structure, not alter its working mode and use materials compatible with those already existing.

Sometimes repair and strengthening options are limited or their execution complicated by the presence of previous works on the bridge. The response of the bridge to past works and their success can provide useful information for assessing the potential effects of the proposed works and their chances of success.

Table C.1 lists a number of commonly used strengthening techniques for masonry arch bridges. The methods presented in Table C.1 and other repair techniques are discussed in further details in SB 6.2 (2007).

Table C.1. Overview of existing strengthening methods for Masonry Arch Bridges

Method	Summary	Structural defect and location	Advantages	Disadvantages	Performance	Experience with existing railways
Concrete saddle	Replacement of existing fill material with a reinforced concrete saddle, to which the spandrel walls and extrados are sometimes stitched using structural ties, aiming to create a composite structure with enhanced stability and to facilitate waterproofing repairs.	Inadequate overall load carrying capacity of arch barrel in conjunction with spandrel wall and waterproofing failures.	• no change to appearance as hidden • facilitates other re pairs/parapet upgrades/waterproofing • enhanced live load capacity	• traffic disruption during construction • relative cost • increase in crown depth	Effective implementation and inspection/maintenance will enhance structural performance in line with the strengthening or repair design life.	Used
Prefabricated liners	Structural lining (normally corrugated steel or precast concrete liners) are installed beneath the existing arch structure to provide a secondary support mechanism within an existing deformed or deteriorated arch	Inadequate overall live load carrying capacity of arch and/or abutments where depth of fill over the arch barrel is excessive. This can also address spandrel wall and waterproofing failures.	• no change to appearance as hidden • facilitates other repairs • enhanced live load capacity	• traffic disruption during construction • relative cost • increase in crown depth	Existing structure assumed to be redundant with liner designed to take full dead and live loading.	Used
Retro- reinforcement	Installation of additional structural reinforcement to the arch barrel aims to increase its structural capacity while not reducing structure clearances or significantly affecting the bridge's appearance.	Inadequate overall load carrying capacity of arch barrel.	• repairs hidden • much less disruption than saddle/slab/ Reconstruction • relative cost • speed of implementation	• Independent verification/validation of analysis, design, installation, fatigue and durability of systems	Effective implementation will allow the structure to support specific enhanced loadings	Some railway authorities do not accept this method
Relieving slab	Installation of a horizontal reinforced concrete slab over the plan area of the arch, extending over the abutments. Aims to improve live load carrying capacity of the arch while eradicating the generation of additional horizontal thrust from the arch into the abutments at springing level.	Incompetent existing fill material or inadequate overall load carrying capacity	• no change to external appearance • enhanced live load capacity	• traffic disruption during construction • relative cost • increase in crown depth possible	Effective implementation will allow the structure to continue to perform as originally designed with increased capacity.	Used
Sprayed concrete lining	Application of structural sprayed concrete to the arch barrel intrados to repair and strengthen arches which are suffering from major defects such as arch barrel distortion, deteriorated masonry and severe cracking.	Inadequate overall carrying capacity	• little disruption to traffic flow over the bridge • enhance load carry capacity • reinforcement can be incorporate	• alter appearance • reduces opening under the bridge • cannot inspect condition of original arch barrel	Spayed concrete provides strengthening mechanism for weakened deteriorated structures.	Used
Thickening surfacing	Provision of an additional thickness of surfacing distributes the live loads more evenly through the arch and can result in higher live load capacity for the structure.	Inadequate overall live load carrying capacity of arch barrel	• possible enhanced live load capacity • relative cost	• traffic disruption during construction • structure life expectancy unaffected by works • further maintenance works may be required	Improved performance and capacity of original structure	Used

Appendix D - Strengthening of the subsoil

Many in-situ site improvement technologies are available to increase the stability and mitigate settlements in bridge transition zones. Several technologies have been used to support new embankments in railway transition zones, such as embankment piles with pile caps, pile decks, deep mixed columns, and the construction of embankments using light weight fill.

However, many currently available methods for strengthening the subsoil adjacent to existing railway bridges require railway traffic to be interrupted. These methods often require that the railway tracks and sleepers be removed, and the ballast and embankment fill be excavated in order to perform the strengthening works. Such methods are complicated, time consuming and expensive. Ideally, soil improvement works should be performed with minimal impact on the railway traffic and without, or with only marginal, reduction of train speed and axle loads. There is a need for methods that can be used to improve the subsurface soils within the transition zone of existing railways. Table D.1 presents an overview of such soil improvement methods; the methods presented in Table D.1 are discussed in greater detail in SB-STR 82007) and SB 6.2 (2007).

The methods presented in Table D.1 were chosen as most relevant with reference to the:

- strengthening effects possible to achieve,
- influence on the existing railway,
- applicability for European soil and railway conditions and
- availability in Europe.

The costs for strengthening measures at existing transition zones are very site specific and have different relative costs in different countries. The costs provided in Table D.1 are approximate values for the Nordic countries.

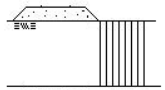
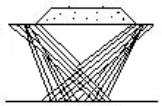
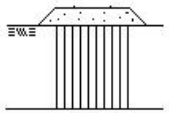
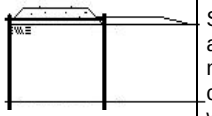
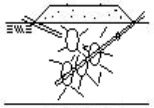
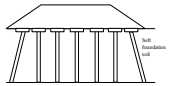
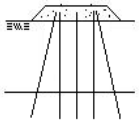
Prior to choosing the appropriate soil improvement method, a geotechnical investigation should be performed to determine the soil stratigraphy and soil material property values at the project site, including the actual soil property values below the embankment. The investigation should also determine the depth to water table and if there are any buried obstructions beneath or in the vicinity of the railway.

For all subsoil strengthening methods, it is recommended that an experienced contractor be involved in the design process, especially for the “specialty” technologies such as deep mixing or grouting. During and after installation of any subsoil strengthening work, the following should be closely monitored:

- Vertical movements, including settlement and heave,
- Horizontal movements,
- Railway track geometry, and in some cases,
- Vibrations, and
- Pore pressures.

Monitoring programs will be project specific.

Table D.1. Overview of existing strengthening methods of subsoil in transition zones at existing railway bridges.

	Method	**Principle**	**Can be performed without affecting traffic**	**Applicable soils**	**Increases Stability**	**Reduces Settlements**	**Approximate Costs**	**Experience with existing railways**
	Deep Mixing, beside railway embankment	Mixes in-situ soils with cementitious materials to form a vertical stiff inclusion in the ground	Yes	Wet method: most soft soil types; Dry method: soft fine-grained soils	X		10 Euro/m column at an amount of binder of 90-120 kg/m^3	Some
	Deep Mixing, installed inclined under embankment	Mixes in-situ soils with cementitious materials to form an inclined stiff inclusion in the ground	Yes	Wet method: most soft soil types; Dry method: soft fine-grained soils	X	X	15 Euro/m column at an amount of binder of 90-120 kg/m^3	None (some for road embankment)
	Deep Mixing, installed through embankment	Mixes in-situ soils with cementitious materials to form a vertical stiff inclusion in the ground	No (Yes, if performed during periods with no traffic)	Wet method: most soft soil types; Dry method: soft fine-grained soils	X	X	40-100 Euro/m column Depending on access	Some
(Same three configurations as for Deep Mixing)	Jet grouting	Erodes soil in situ and mixes with cementitious materials to form stiff inclusion in the ground	Yes (unless installed beneath embankment)	Most soil types	X	X	Mob/Demob 30-50,000 Euro 250-350 Euro/m column	Some
	Stabilizing berms, alone or in combination with anchored sheet pile walls	Compacted material constructed adjacent to embankment. Driven steel sections provide resistance against horizontal movements.	Yes	Clay	X		150-200 Euro /m^2 sheet pile excl. anchoring	Extensive
	Compaction grouting	Low slump grout is pumped into the ground to form grout bulbs, which displace and densify the soil	Yes	Granular soils	X	X		Some
	Pile deck or piles with pile caps and possibly, geosynthetic reinforcement	Piles transfer loads to more competent strata through friction or end-bearing; piles caps and geosynthetic transfer load to piles	No	All soil types	X	X	For driven concrete piles: 45-55 Euro/m + Pile caps	Extensive
	Embankment piles, without pile caps	Driven or grouted piles transfer loads to more competent strata through friction or end-bearing	No (Yes, if performed during periods with no traffic)	All soil types	X	X	For driven concrete piles: 45-55 Euro/m + Pile caps	Some

Appendix E – Method Descriptions

The following Method Descriptions are available:

MD0	**Concrete Bridges**
MD001	Adhesively Bonded CFRP Plates to Concrete Structures
MD002	Adhesively Bonded CFRP Sheets to Concrete Structures
MD003	Strengthening of Concrete Structures with Mineral Based Composites
MD004	Near Surface Mounted Reinforcement (NSMR) to Concrete Structures
MD005	External Prestressing of Concrete Structures

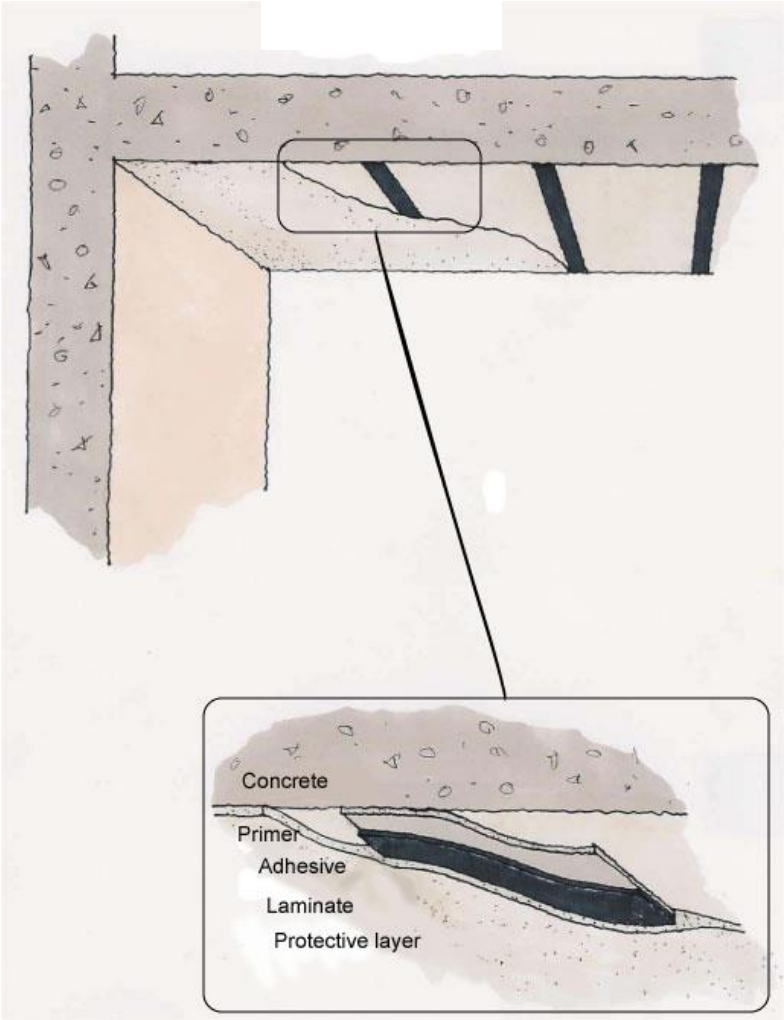
MD1	**Metallic Bridges**
MD101	Adhesively Bonded CFRP Plates to Metallic Structures
MD102	Adhesively Bonded CFRP Sheets to Metallic Structures
MD103	External Prestressing of Metallic Structures
MD104	External Prestressed CFRP Plate

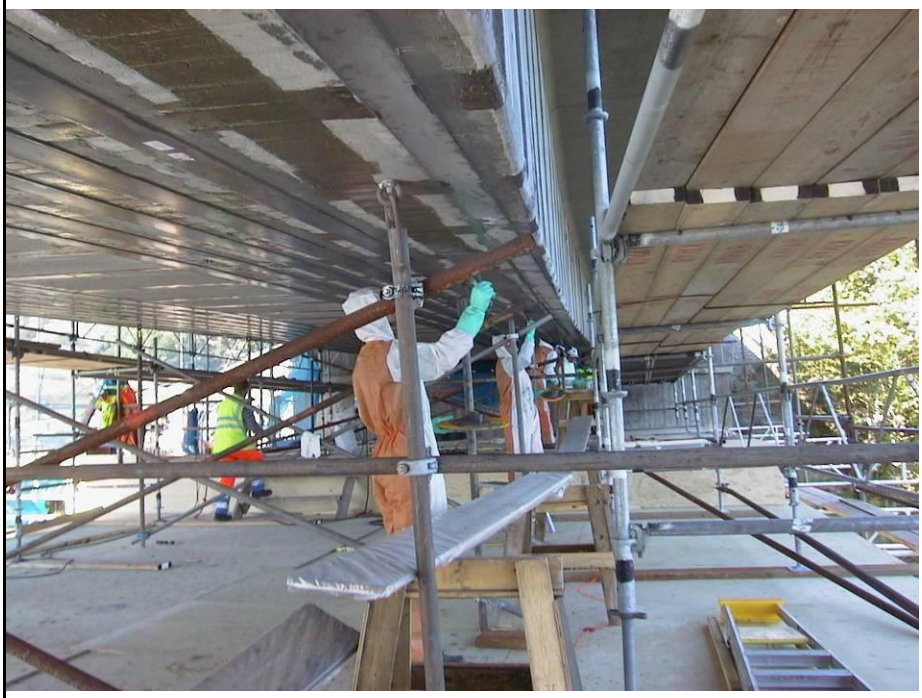
MD2	**Masonry Bridges**
MD201	Adhesively Bonded FRP Plates to Masonry Structures
MD202	Adhesively Bonded FRP Sheets to Concrete Structures
MD203	Strengthening of Concrete Structures with Mineral Based Composites
MD204	Near Surface Mounted Reinforcement (NSMR) to Masonry Structures
MD3	**Subsoil and Foundation**
MD301	Deep Mixing
MD302	Jet Grouting - Subsoil
MD303	Sheet Pile Walls/Stabilising Berms
MD304	Compacting Grouting
MD305	Embankment Piles
MD306	Jet Grouting - Foundation
MD307	Compaction Grouting
MD308	Shaft Grouting and Base Grouting

Methods MD001, MD002 and MD101 are given below as examples. The others can be found in SB-STR (2007). :

MD001 **Date:** 2014-05-05 **Method:**	**Adhesively Bonding of CFRP Plates to Concrete Structures**	
Objectives: Type of rehabilitation	[Check appropriate Box(es)] ☒ Structural Repair ☒ Structural Upgrading ☐ Safety and Comfort	
Field of application	[Tick appropriate box] ☒ Concrete ☐ Metal ☐ Masonry ☐ Subsoil ☐ Foundation	[For what problem is the method suitable, see graphical index] A, C
General description	[General description of the rehabilitation method] Structural concrete members with limited load carrying capacity in need of upgrading could improve its capacity by additional reinforcement in form of external bonded FRP plates. The plates do not need much space and are placed on the surface of the member. They are non-corrosive, light yet stiff and strong and can be delivered on rolls up to approximately 200 m. The stiffness properties can be varied from approx. 150 – 400 GPa. Mostly the plates are used for flexural strengthening in the ultimate limit state, but can also be used for shear strengthening and in situations when need of minimising deflections or concrete cracking, i.e. in the service limit state. Special design provision yields.	
Work description:	[Work description of the rehabilitation method] The work can be divided into three stages, before strengthening, during strengthening and after strengthening. However, before the strengthening work starts a detailed investigation of the existing structure must be carried out. This investigation may vary from structure to structure and depend on the need. This is not further discussed here. Before strengthening the concrete surface is properly cleaned from any grease or oil debris. The ballast shall be uncovered to a size of approximately one square centimetre. Normally sandblasting, grinding or both is used. The dust shall then be removed before the bonding procedure starts. During strengthening, depending on the strengthening system used, the areas to be bonded are treated with a primer to enhance the bond for the epoxy adhesive. The plates are usually provided with a peel-ply (plastic protection sheet) that is removed at time for bonding. To the plates a two component epoxy adhesive is applied by an “adhesive tool”. This tool is tailored for the plate to be bonded. The adhesive thickness should be less than 1.0 mm and the maximum thickness 3 mm, locally the thickness might be larger. Temperature at bonding must exceed 10 °C. The plate is then mounted to the concrete surface and given a light pressure to squeeze out possible air bubbles. The plates can come in many different dimensions; a normal thickness is 1.0 to 2.0 mm and the width 50 – 200 mm. Excess adhesive is scraped off. After strengthening the peel ply on the outside is removed. The boding is then investigated with consideration to possible voids. This might be done with a light tapping on the surface with a coin or a small hammer or with more advanced methods such as thermography.	

Traffic regulations	[Are special traffic management needed, if so, what?] The method has proven to work with thermo setting polymer as adhesive with live loads acting on the structure during the strengthening process. During strengthening work, access is needed to the parts that are going to be strengthened. Critical work can be access to the underpass
Prerequisites	[What factors limit the method?] The concrete quality should normally exceed 30 MPa in compression and the pull-off strength should exceed 1.5 MPa. In addition larger irregularities should either be grinded off or levelled out.
Environmental considerations	[What special environmental impact must be considered, i.e. aesthetics, contaminants etc.] When working with epoxy adhesive stipulated regulations must always be followed. Skin contact shall be avoided and prescribed protective clothing must be used. Special regulation must also be followed when non hardened epoxy products is going to be taken care of.
Costs Construction Traffic management Maintenance	[Estimate the cost levels for the method] ☐ High ☒ Medium ☒ Low ☐ High ☐Medium ☒ Low ☐ High ☐Medium ☒ Low
Equipment and materials	[Required technology] No special equipment is needed except equipment for bonding the plates. Very important for the final strengthening result is proper scaffolding. In particular in areas where it otherwise can be complicated to reach the components that needs to be strengthened.
Codes and standards	[Can existing codes and standards be used, or is there a need of new guidelines] Most countries has national guidelines for external strengthening. However, no common guideline exists today for Europe.
Pros and cons	[Important advantages and disadvantages] <u>Pros:</u> The materials are very light, yet strong and stiff, they are easy to apply and can hence be used for many different type of concrete components. Cost effective. Even tough the material itself in comparison is expensive, when taking the production cost in consideration, the method is very competitive compared to alternative strengthening methods. The CFRP material is very environmental stable and does not corrode. <u>Cons:</u> Environmental regulations due to the epoxy adhesive have to be followed and protective clothing must be carried. The systems are temperature and moisture sensitive at time for application. Experience over long time is scares. The knowledge how to design for strengthening is not wide spread among the consultancies and the same yields for the contractors where strengthening skills is lacking,

Alternative methods	[Alternative methods to be considered in the design stage] Alternative methods depend on the particular strengthening need. Alternative methods might be: • External prestressing • External bonded steel plates • Reinforcing and casting of a additional concrete layer
Comments	[Additional information related to the system] It needs to be stressed that the strengthening work on site is very important for the final strengthening results. That means that the workers must be skilled and have experience from external strengthening work with bonded plates. Furthermore, when working with thermosets the surrounding environment is very important. That means that the work has to be carried out at a minimum temperature and that the relative humidity must be below a certain threshold. Additionally, the strengthening result is very dependent on the condition of the existing structure, If weak layers exist this must be corrected. Normally corrosion at the internal reinforcement is not allowed.
Sketches	[Illustration to support the description of the method on general basis] 

Photos	[Photos to support the description of the method on general basis] Before Strengthening  Sandblasting
	During strengthening  Priming of the concrete surface

	 Application of adhesive
	 Strengthening works

	After strengthening  Final result – painted surface

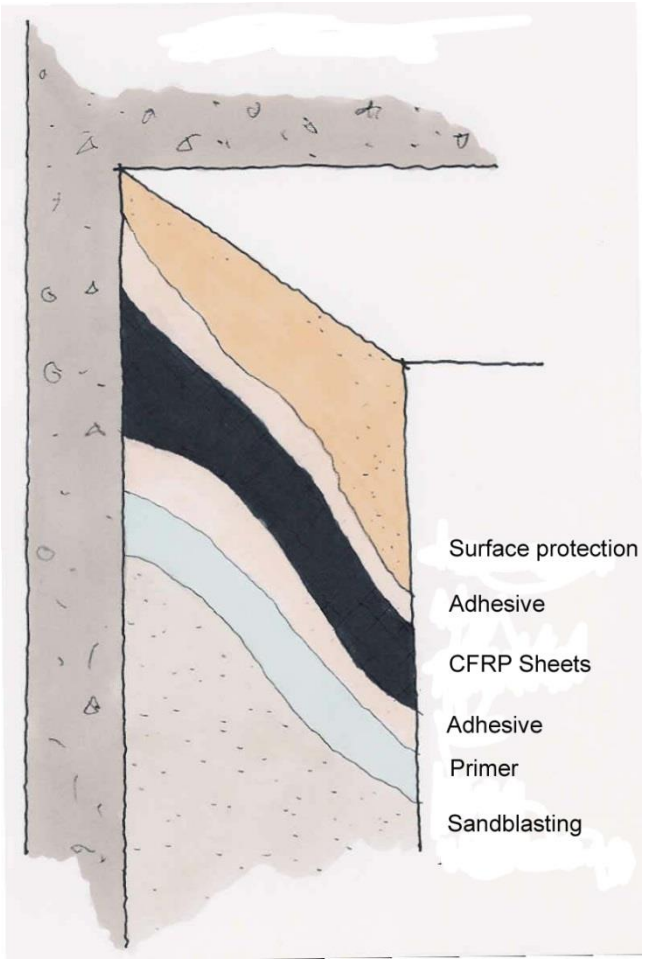
MD002 **Date:** 2014-05-05 **Method:**	**Adhesively Bonding of CFRP Sheets to Concrete Structures**	
Objectives: Type of rehabilitation	[Check appropriate Box(es)] ☒ Structural Repair ☒ Structural Upgrading ☐ Safety and Comfort	
Field of application	[Tick appropriate box] ☒ Concrete ☐ Metal ☐ Masonry ☐ Subsoil ☐ Foundation	[For what problem is the method suitable, see graphical index] A, C
General description	[General description of the rehabilitation method] Structural concrete members with limited load carrying capacity in need of upgrading could improve its capacity by additional reinforcement in form of external bonded FRP sheets. The sheets do not need much space and are placed on the surface of the member. They are non-corrosive, light yet stiff and strong and can be delivered on rolls up to several hundred meters. The stiffness properties in the fibre direction can be varied from approx. 200 – 600 GPa on the fibre The sheets are mostly used for shear or confinement strengthening in the ultimate limit state, but can also be used for flexural strengthening and in situations when need of minimising deflections or concrete cracking, i.e. in the service limit state. Special design provision yields.	

Work description:	[Work description of the rehabilitation method] The work can be divided into three stages, before strengthening, during strengthening and after strengthening. However, before the strengthening work starts a detailed investigation of the existing structure must be carried out. This investigation may vary from structure to structure and depend on the need. This is not further discussed here. Before strengthening the concrete surface is properly cleaned from any grease or oil debris. The ballast shall be uncovered to a size of approximately one square centimetre. Normally sandblasting, grinding or both is used. The dust shall then be removed before the bonding procedure starts. When using sheets it is important the surface does not contain any larger irregularities. The fibres must be straight to be most efficient. During strengthening, the areas to be bonded are treated with a primer to enhance the bond for the epoxy adhesive. The sheets are usually provided with a protective paper or plastic film that is removed at time for bonding. To the primed surface is a two component low viscosity adhesive applied with a mohair roller, in the adhesive is then the fibre sheet mounted. The fibres are stretched and a additional layer of adhesive is applied. This procedure may be repeated up to approximately 10 layers of sheets. Temperature at bonding must exceed 10 °C. The normal thickness of the final composite is 0.5 mm up to 10 mm, depending on number of layers. After strengthening the bonding is then investigated with consideration to possible voids. This might be done with a light tapping on the surface with a coin or a small hammer or with more advanced methods such as thermography. The surface may also be covered with a protective layer in form of plaster, paint or concrete.

Traffic regulations	[Are special traffic management needed, if so, what?] The method has proven to work with thermo setting polymer as adhesive with live loads acting on the structure during the strengthening process. During strengthening work, access is needed to the parts that are going to be strengthened. Critical work can be access to the underpass
Prerequisites	[What factors limit the method?] The concrete quality should normally exceed 30 MPa in compression and the pull-off strength should exceed 1.5 MPa. In addition larger irregularities should either be grinded off or levelled out.
Environmental considerations	[What special environmental impact must be considered, i.e. aesthetics, contaminants etc.] When working with epoxy adhesive stipulated regulations must always be followed. Skin contact shall be avoided and prescribed protective clothing must be used. Special regulation must also be followed when non hardened epoxy products is going to be taken care of.
Costs Construction Traffic management Maintenance	[Estimate the cost levels for the method] ☐ High ☒ Medium ☒ Low ☐ High ☐ Medium ☒ Low ☐ High ☐ Medium ☒ Low

Equipment and materials	[Required technology] No special equipment is needed except equipment for bonding the psheets. Very important for the final strengthening result is proper scaffolding. In particular in areas where it otherwise can be complicated to reach the components that needs to be strengthened.
Codes and standards	[Can existing codes and standards be used, or is there a need of new guidelines] Most countries have national guidelines for external strengthening. However, no common guideline exists today for Europe.
Pros and cons	[Important advantages and disadvantages] <u>Pros:</u> The materials are very light, yet strong and stiff, they are easy to apply and can hence be used for many different type of concrete components. Cost effective. Even tough the material itself in comparison is expensive, when taking the production cost in consideration, the method is very competitive compared to alternative strengthening methods. The CFRP material is very environmental stable and does not corrode. <u>Cons:</u> Environmental regulations due to the epoxy adhesive have to be followed and protective clothing must be carried. The systems are temperature and moisture sensitive at time for application. Experience over long time is scares. The knowledge how to design for strengthening is not wide spread among the consultancies and the same yields for the contractors where strengthening skills is lacking. The use of sheets give normally a lower quality composite compared to plates or NSMR.

Alternative methods	[Alternative methods to be considered in the design stage] Alternative methods depend on the particular strengthening need. Alternative methods might be: • External prestressing • External bonded steel plates or laminates for flat surfaces • Reinforcing and casting of a additional concrete layer
Comments	[Additional information related to the system] It needs to be stressed that the strengthening work on site is very important for the final strengthening results. That means that the workers must be skilled and have experience from external strengthening work with bonded plates. Furthermore, when working with thermosets the surrounding environment is very important. That means that the work has to be carried out at a minimum temperature and that the relative humidity must be below a certain threshold. Additionally, the strengthening result is very dependent on the condition of the existing structure, If weak layers exist this must be corrected. Normally corrosion at the internal reinforcement is not allowed.
Sketches	[Illustration to support the description of the method on general basis]


Surface protection
Adhesive
CFRP Sheets
Adhesive
Primer
Sandblasting

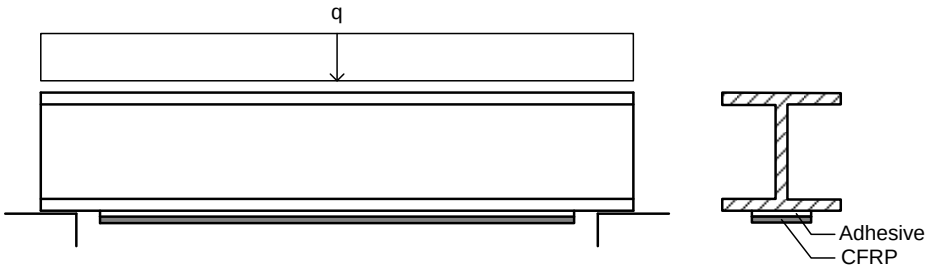
Photos	[Photos to support the description of the method on general basis] Before Strengthening  Sandblasting
	 Larger irregularities are levelled out with epoxy putty
	 Grinding
	During strengthening

	 Priming of the concrete surface
	 Cutting of sheets to prescribed dimensions(length)
	 Application of adhesive to primed surface

	 Mounting of sheets
	After strengthening  The underside has been painted
	 Final result
MD101	[Name of Rehabilitation Method]

Date: 2007-03-05 **Method:**	**Adhesively bonding of Carbon-Fibre-Reinforced-Polymers (CFRP) plates on Metallic Structures**	
Objectives: Type of rehabilitation	[Check appropriate Box(es)] ☒ Structural Repair ☒ Structural Upgrading ☐ Safety and Comfort	
Field of application	[Tick appropriate box] ☐ Concrete ☒ Metal ☐ Masonry ☐ Subsoil ☐ Foundation	[For what problem is the method suitable, see graphical index] G, J, L
General description	[General description of the rehabilitation method] Provision of an additional degree of bending strength in structural elements in order to obtain increase of load capacity in ULS. The CFRP plate has to be applied so the area in tension is upgraded.	
Work description:	[Work description of the rehabilitation method] **Cleaning of metal surface** To remove any dust, paint or unsound material on the bonding surface the steel member has to be mechanically cleaned. Recommended methods are sandblasting (SA2$^1/_2$) or SACO treatment. The former mechanical surface preparation is accomplished by a chemical cleaning of the bonding surface to remove dirt and debris using acetone or any other appropriate cleaning agent. **Application of primer** To optimize the adherence between metal and adhesive and to avoid any corrosion of the substrate, the metal surface has to be treated with a thin layer of primer. This special coating is usually based on the later on applied adhesive and forms part of the bonding system. Therefore the used primer has to be chosen according to the recommendation of the adhesive producer to assure the maximum adherence between the steel and adhesive. **Cleaning of CFRP plate** Depending on the used strengthening system the CFRP surface may be cleaned with acetone before bonding. Some producers protect the plate surface with a special protection layer, which has to be removed before bonding. In such case cleaning of CFRP plate before bonding may not be necessary. (See manufacturer information) **Application of the adhesive** After curing of the primer the adhesive can be applied. To reach a high homogeneity of the adhesive layer, without any air pockets, the adhesive should be applied single sided (preferably on the CFRP side) and in a shape, which forming a triangular cross-section of the adhesive. **Assembling** Once the adhesive has been applied, the CFRP plate has to be placed during the pot-life time of the adhesive. Like all steps before, the assembling has to be carried out with great care to avoid any contamination or air pockets. Therefore the CFRP plate should be attached carefully in one working step, which means that an adequate number of skilled labours and fixing equipments have to be available. Any detaching or replacing may cause air inclusions or con-	

	tamination of the adhesive layer.
Traffic regulations	[Are special traffic management needed, if so, what?] The preparation work of the beam element have no or minor influence on the bride deck and hence no or minor influence on the traffic of the bridge. During the application and curing of the CFRP plates the traffic on the bridge needs to be restricted to ensure good quality of the bond line. The influence on the traffic on the underpass due to the preparation and application work is dependent of the design of the scaffold.
Prerequisites	[What factors limit the method?] The surface of the metallic beam has to be plane and sufficient space must be available for the CFRP plate. The temperature and the humidity at time for application must be in the range according to the instructions given by the manufacturer of the adhesive.
Environmental considerations	[What special environmental impact must be considered, i.e. aesthetics, contaminants etc.] None
Costs Construction Traffic management Maintenance	[Estimate the cost levels for the method] ☐ High ☒ Medium ☐ Low ☐ High ☐Medium ☒ Low ☐ High ☐Medium ☒ Low
Equipment and materials	[Required technology] **Preparation work** Sandblasting equipment, acetone and cleaning tools. **Application and assembling** CFRP plates, adhesive, primer and application tools.
Codes and standards	[Can existing codes and standards be used, or is there a need of new guidelines] Guidelines for strengthening of metallic structure with CFRP plates are available.
Pros and cons	[Important advantages and disadvantages] <u>Pros:</u> Improved capacity in ULS with only small increases in self-weight. No stresses are introduced due to application or reductions of sectional areas. The application is fast executed compared to traditional methods. The strengthening system is not sensitive to fatigue damage or corrosion. Low influence on the aesthetics. <u>Cons:</u> The failure of the strengthening system is brittle. High demands are request on both the surface preparation and application, i.e. high skilled labours. CFRP plates are sensitive for impacts, i.e. carefulness has to be taken when handling the material on site. The long-term effects are not well documented.
Alternative methods	[Alternative methods to be considered in the design stage] - Pre-tensioned CFRP plate. - Application of extra steel material. - Post-tensioned cables.

Comments	[Additional information related to the system] The strengthening system has to be covered by a surface protection coating to prevent degradation due to environmental exposure (e.g. de-icing salt in combination with high humidity)
Sketches Principle sketch of the system (side view and cross-section)	[Illustration to support the description of the method on general basis] 
Photos Cleaning of the metal surface by blasting	[Photos to support the description of the method on general basis] 
Application of primer	

Cleaning of CFRP plate	
Application of the adhesive	
Assembling	

Appendix F – Case Studies

A few case studies can be found in the references given below. Case study CS002:01 “CFRP sheets, Kallkällan” is also given below as an example. More case studies are given in SB 6.2 (2007).

CS0	**Concrete Bridges**	**Reference**
CS001:01		
CS001:02	CFRP laminates and sheets, Källesund	CS001:02 in SB-STR (2007)
CS002:01	CFRP sheets, Kallkällan	CS002:01 in SB-STR (2007)
CS003:01		
CS004:01	Near Surface Mounted Reinforcement, Örnsköldsvik	SB 6.3 (2007), Puurula et al (2012, 2015)
CS005:01	External Prestressing, Woodland	CS005:01 in SB-STR (2007)

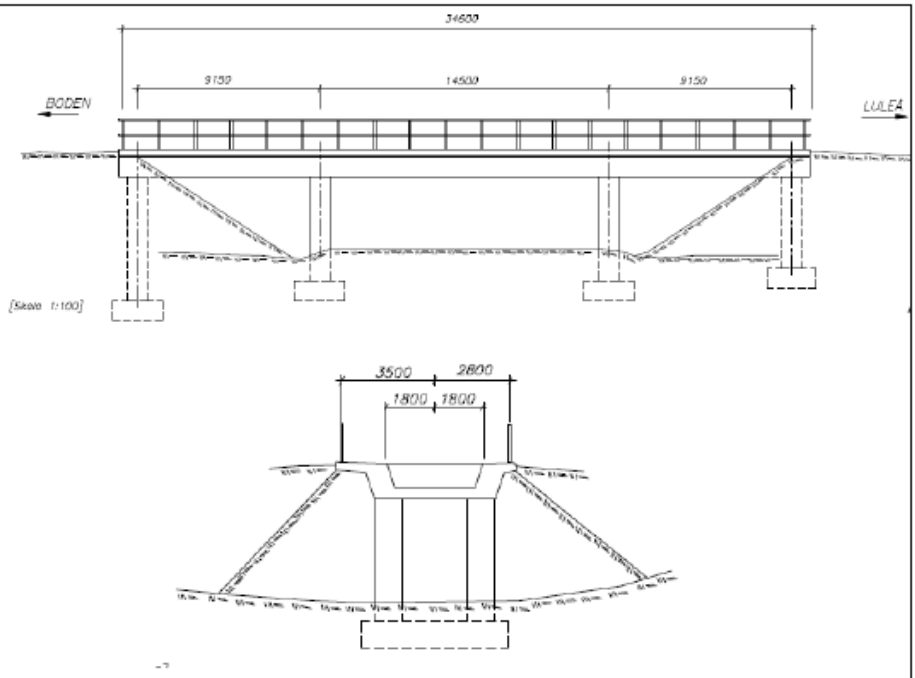
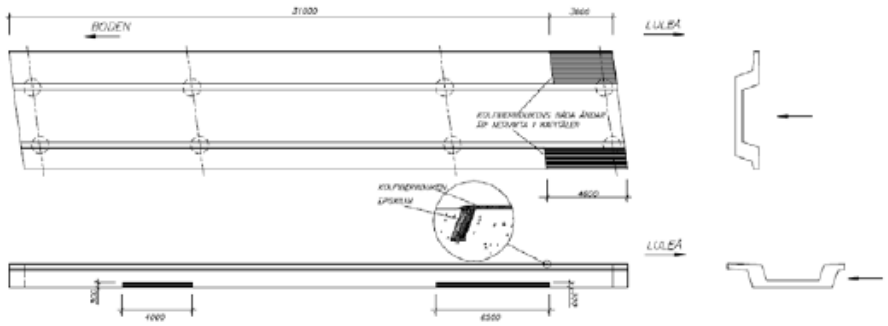
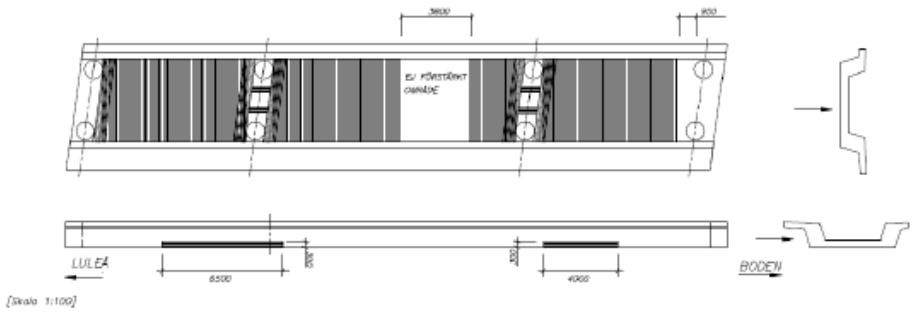
CS1	**Metallic Bridges**	
CS101:01	I-beam bridge with CFRP plates	CS101:01 in SB-STR (2007)
CS101:02	Post-tensioned cables and new rivets	CS005:02 in SB-STR (2007)

CS2	**Masonry Bridges**	
CS201:01	Strengthening by Concrete Saddle	CS201:01 in SB-STR (2007)

CS3	**Subsoil and Foundation**	
CS301:01	Sheet Pile Walls	CS301:01 in SB-STR (2007)

CS002:01 **Date:** 20070518	Sustainable Bridges
Method	**Adhesively bonded CFRP sheets to concrete strucutres**
Project definition	Strengthening of railway concrete trough bridge in Luleå Sweden. The bridge was strengthened in 1998 with CFRP Sheets
Structural elements to be rehabilitated	A railway concrete trough bridge in two spans needed to be strengthened in the cross direction, mainly the bottom slab was strengthened.
Main objectives	To meet the updated requirements stated by the bridge owner: - to increase the existing axle load from 25 tons per axle to 30 tons per axle due to increased loading from the transport of iron ore.
Rehabilitation works	Strengthening with CFRP involves three main moments; preparation work before strengthening, strengthening work and finishing work. Preparation work, includes theoretical calculations as well as all work before bonding the sheets. The finishing work in this particular case refers to protective painting. <u>Preparing work – before strengthening</u> The load conditions have been calculated by the Swedish Road Administration. These calculations gave the need for strengthening the bridge in bending in the bottom flange in the cross direction. There where also a need to strengthen the bridge with respect to shear tension in the section between the web and the flanges in the bottom as well in the top of the bridge. In some sections it where also a need to strengthen the bridge with consideration to short anchor lengths of the internal steel reinforcement. However, in this case study only the strengthening in the cross direction of the bottom flange will be reported. To ensure sufficient adhesion between the CFRP sheets and the concrete low quality concrete must be removed and the aggregates must be uncovered. In addition, the concrete surface must be even enough that the carbon fibre sheets will straight, These two demands implies sandblasting and often also grinding, see also Fig.1. For the underpass at Kallkällan both sandblasting and grinding was used. Sandblasting to uncover the aggregates and grinding to smoothen out the fins in the concrete from the formwork when the slab was cast. The surface was thereafter cleaned with compressed air to remove dust and debris. <u>During strengthening</u> Before the bonding procedure started a two component diffusion open primer was applied to the concrete surface. The purpose with the primer is to increase the bond between the CFRP sheets and the concrete and to prevent the epoxy adhesive to penetrate into the concrete without wetting the fibre sufficiently. Approximately one day passed before the adhesive was applied on the primed surface. At larger irregularities epoxy putty was used to level out the concrete surface. The two component epoxy adhesive was applied wet-in wet in the putty and then the CFRP sheets were mounted. All sheets had been pre-cut before mounting.

	The CFRP sheets was mounted in the wet adhesive and possible air was squeezed out with a mohair roller as shown in Fig. 2 In total 3 layers of sheets was applied. After each CFRP sheet a new layer of epoxy adhesive was applied until all layers have been mounted. In Fig. 3 mounting of the CFRP sheets are shown. After Strengthening After strengthening the strengthen surface was protected with a two component polyurethane paint. This particular paint is designed for outdoor use and gives a very hard and though surface similar to surfaces in industry buildings. It will also protect the strengthening system against UV-radiation. In this particular case also a full scale test was carried out before and after strengthening. This test showed that the decrease in steel stresses and deflections was approximately 17 % after strengthening. This was obtained using just 3 layers of CFRP sheets. Additional information The concrete average compressive strength was 61.3 MPa and the pull-off strength of the concrete 3.0 MPa, both from three tests. The CFRP Sheets had a weight of 300 g/m^2 and a modulus of elasticity of 235 GPa. The strain at failure was 1.5 %. The volume fraction of the final composite on site was measured to approximately 38 %.
Traffic Management	Traffic management was needed since there was an road underpass under the bridge. One line had to be closed during the strengthening work.
Standards and Codes	No standards were available at time for strengthening.
Relevant alternatives	In this particular case no relevant alternatives was considered useful for the strengthening work
Comments	Epoxy is a thermosetting plastic that consists of two parts, a resin and a hardener. The fully cured product involve no environmental or health problems and are often used as package of food. On the other hand if the two compounds are improperly handled they can cause allergy and irritations. Therefore, it is important that the epoxy is handled as prescribed by the local authorities. In this particular case the epoxy was mixed at a special "mixing station" at site where possible waste was taken care of immediately. Un-cured epoxy was placed in special barrels that was sent for destruction when the strengthening work was finished. Almost 10 years after the strengthened work was completed the strengthening works as intended. There have been some minor damages due to vehicle impact and in one area the protective paint had fallen off. The reason for this was that the contactor in this specific area had not followed the instructions for painting, i.e. the surface had not been treated in the correct way.

Drawings	 Longitudinal and cross-direction of the Kallkällan Bridge.
	 Strengthened bridge seen from the top and from the west side
	 Strengthened bridge seen from the bottom and the east side

Photos	 Fig. 1 Concrete surface before applying the fibres. Larger irregularities have been filled with a epoxy putty. Here the corners are grinded off.
	 Fig. 2 Excess air is squeezed out by a roller. The release paper is removed before the next sheet is applied
	 Fig 3. Mounting of CFRP sheets

Fig. 4 The bottom of the slab has been painted

Fig. 5 Finished strengthening work.

Construction Period	Start: June 1998	End: August 1998
Client	Banverket - Sweden	
Designer	Skanska Teknik AB and Luleå University of Technology	
Contractor	Stabilator AB (Spännarmering AB)	

Appendix G – Design of Strengthening

G.1 INTRODUCTION

G.1.1 General

There is an increased need the last decades to maintain, repair and upgrade our existing civil structures and buildings. The increase can be related to impaired performance. Performance relates here to load carrying capacity, technical function, durability or aesthetics. There are at least two strong arguments why an existing concrete structure shall be repaired or upgraded instead of replacing it with a new one. Firstly; financial reasons – it is almost always economical to repair or upgrade a structure then demolishing and constructing a new. Secondly; environmental reasons. By extending the life of the structure the use of our natural resources are lowered. There might also be third reason; In general the time to upgrade an existing structure is considerably shorter than building a new. In addition it is often also possible to keep the structure in use during repair/upgrading. If focus is placed on insufficient load carrying capacity can the reason for this be divided into two main areas:

1. Change in use; the structure have to take other loads than it was originally designed for
2. Degradation of material; the structure has deteriorated to a level that it cannot carry the loads it was designed for.

G.1.2 Change in use

Normally building structures should have a long life, for example a civil engineering structure, e.g. abridge, is estimated or designed for having a life span extending 100 years. It is then understandable that the demands or even the usage might change over time. The structures do not always have the extra safety to meet these changes and different measures, for example strengthening, might be needed. Such underlying causes might have the bases related to the following:

- Mistakes made in the planning or execution phase
- Demands related to increased load carrying capacity or safety
- Change of function of the structure, e.g. in relation to reconstruction

I all these cases FRP:s for strengthening can in general be used.

G.1.3 Degradation of material

Degradation of concrete structures can be caused by many different physical and chemical processes. Common deterioration processes are induced by chlorides from de-icing salts or sea water or through carbonatisation of the concrete cover. Both these mechanisms cause steel corrosion. Other factors that might negatively affect a concrete structures load carrying capacity are freeze-thaw damages and lime leakage that impair the strength of the concrete and in some cases leads to considerably lower safety of the structure. The measure might be demolition and building a new structure or by repair and upgrading. What choices considered have to be analyzed from case to case. In situations when the deterioration have not reached critical levels and if the future deterioration can be halted, often FRP:s can be used as a repair or strengthening measure. The suitability to use FRP:s have to be decided by a specialist based on the actual situation.

G.1.4 Strengthening of concrete structures

Before a decision is taken regarding strengthening of concrete structures a proper assessment is recommended to make clear the reason for strengthening. Some common used methods to strengthen concrete elements are to increase the cross sectional area, external prestressing, shotcrete, change of static system etc. These methods normally have a good functionality and have also been used successfully for a long time. In the mid 70-ties a strengthening method using external bonded steel plates was developed. During the 70-ties and 80-ties the method was relatively commonly used in central Europe, US and Japan. During the beginning of the 80-ties the use of FRP (Fibre reinforced Polymers) for strengthening of concrete structures was researched. The primary aims were to find methods and systems to improve the dynamic response on structures in relation to earthquakes. The effect was very positive and continued research in the area led to the development of unidirectional FRP laminates for external strengthening. Today externally strengthening with FRP:s are commonly used and fully accepted around the world. A common denominator for the strengthening systems has been carbon fibre, since this type of fiber have excellent mechanical properties for strengthening purposes. In Sweden research in this area started in the end of the 80-ties at Luleå University of Technology. First with bonded steel plates and in the beginning of the 90-ties with composites. Today continuous research is carried out at the university.

The experience in the field of FRP strengthening is approximately 30-35 years and if also steel plate bonding is considered more than 50-years' experience exists. Early research in this field can consider to have been pragmatic, where focus was placed to understand how to strengthen and how much a structure can be strengthened. Later in the research more refined design models have been developed, evaluated and implemented. So there is a good understanding on how to carry out the strengthening in an appropriate way on a structure, as well as a good understanding how to design for strengthening.

In this section a brief summary of how to design for flexure, shear and confinement is presented. More detailed information is given in Täljsten et al (2006, 2011).

G.2 MATERIALS AND STRENGTHENING TECHNIQUES

G.2.1 General

In contrast to traditional industries where composites have been used over a long time, such as the space, aircraft and car industries, composites in the construction industry must provide for longer lifespan. Existing demands on structural life of 50 years or more are common in the construction industry. Only those systems that have been extensively tested and applied in full-scale on concrete structures are possible candidates to use in external strengthening with FRP's. It is important to treat a proven system as a whole where its function has been verified by tests and applications.

Systems for FRP strengthening can be subdivided into prefabricated systems and in-situ systems. Whereas, prefabricated systems usually refer to pultruded flat profiles or rods, in-situ systems refer to fabrics or sheets that together with a resin forms a composite on site. Here, the systems are described in general terms and for design and strengthening works the supplier's recommendations shall be followed. There are three general steps that should be followed: pre-treatment, strengthening and post-treatment.

- The pre-treatment involves uncovering of aggregates, leveling and cleaning of the surface. No dust, grease or water shall exist on the surface at the moment of bonding.
- The strengthening process depends on the system chosen, but the bonding temperature shall exceed *10°C* to allow the adhesive to harden. For temperatures below *10°C* an external heat source or a heating device must be used.
- Post-treatment can involve fire protection, application of plaster, paint or other protection systems that are deemed necessary.

Different possibilities of strengthening building structures are shown in Figure G.2.1. FRP strengthening is suitable for concrete beams, walls, slabs and columns, but can also strengthen openings in slabs or walls. Another application is to strengthen structural elements by bonding FRP rods in the concrete cover, so-called NSM (Near Surface Mounted) Reinforcement. Questions regarding fatigue are treated in Mahal (2015).

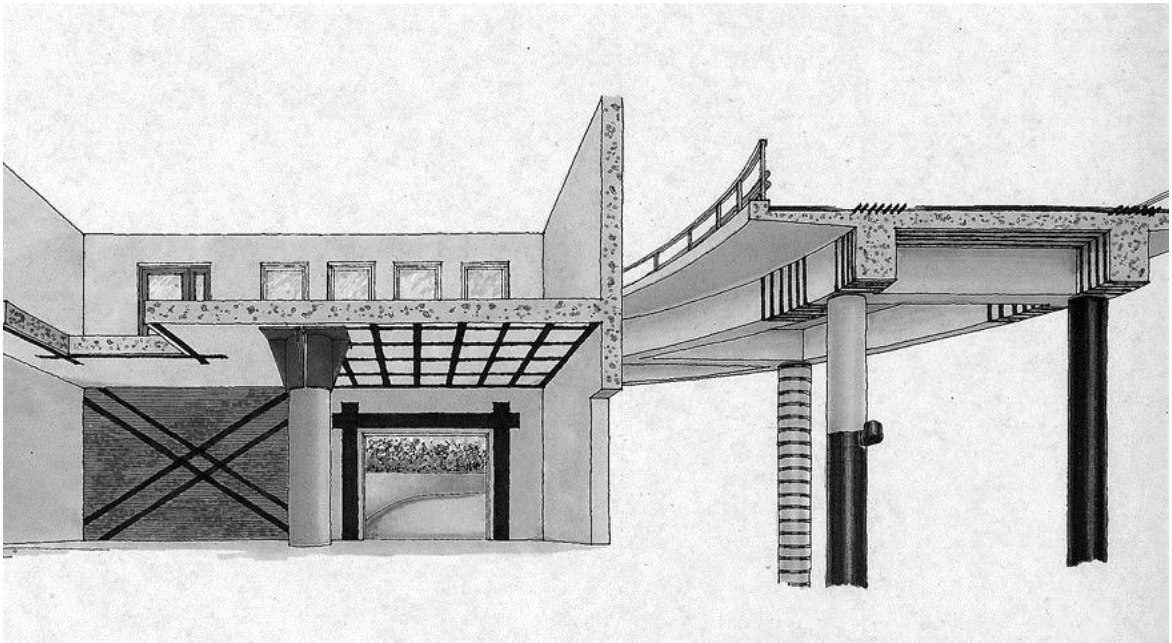


Figure G.2.1 – Strengthening possibilities with plates, sheets and NSM.

G.2.2 Plates

The first applications with CFRP laminate or plate system were carried out in Switzerland during the beginning of the 1990s, where a concrete bridge was strengthened due to an accident that broke pre-stressing cables. Since then a large number of objects have been strengthened worldwide. A plate system consists normally of flat laminates with a typical size of (1.2-1.4) x (50 -150) mm, but also other dimensions are available. The laminates can be obtained in different material grades, but normally the Young´s modulus varies between 150-250 GPa with a strain to failure between 0.4% up to 1.9%. Theoretically, the length of the laminate can be unlimited but practically the length is limited to 20-30 meters. Other components in the strengthening systems are primer and adhesive. The function of the primer is to enhance the bond for the adhesive to the concrete. The adhesive used is a high viscosity filled paste such as epoxy adhesive. A typical bond layer thickness is 1 - 3 mm.

Laminates are most suitable for flat surfaces such as beams, walls and slabs. After the concrete has been pre-treated, the adhesive layer is placed on to the laminate and in some cas-

es also to the concrete surface. The two adherents are then mounted together and a light pressure is applied on the laminate. Thereafter the system is allowed to harden.

G.2.3 Sheets

Sheet systems are usually based on dry unidirectional fabrics, but bi-directional weaves are also used. The sheet systems are more sensitive to the irregularities in the concrete surface and often more pre-treatment is needed. However, the sheet systems are flexible and can be adapted to most surfaces. Sheet systems have found their application in seismic retrofitting and the strengthening of curved structures, such as silos. These types of systems are also very suitable in cases where openings need to be strengthening in walls or slabs. A typical sheet system consists of an epoxy primer, putty, dry or pre-impregnated fibre and a resin system.

Often the post-treatment consists of painting, but also plaster or a thin layer of polymer concrete has been used. The sheets used normally have a width of 200 - 400 mm with a weight of 200 - 400 g/m^2. The strengthening process for sheet systems is a little bit more time demanding than for the laminate system. First, the concrete surface is pre-treated. A primer is then applied and in cases of large unevenness, putty is used to level out these irregularities. The next step is to apply a thin layer of low viscosity epoxy adhesive to the concrete surface and then roll the carbon fibre sheet out over this surface. The fibres are stretched, and a roller is used to press out possible air voids, then a new layer of adhesive is applied. This process can be repeated up to as much as 10 - 15 no. of layers depending on the strengthening system used.

G.2.4 Near Surface Mounted Reinforcement

Near Surface Mounted Strengthening (NSM) systems are used in cases where the strengthening system needs to be protected, for example in the case of possible impact. NSM systems are also suitable to use if the concrete surface is very uneven. Most NSM systems consist of circular or rectangular rods that are bonded in slots in the concrete cover of a structure. It is important to control the thickness of the concrete cover before this method is chosen; a typical concrete cover depth of at least 25 mm is normally needed. The pre-treatment for this method consists of sawing slots in the concrete cover. The rods are then bonded in these slots with an epoxy adhesive or a high quality cement grout.

It is of utmost importance that the slots are cleaned immediately after sawing; all concrete dust, wet concrete or ashes concrete must be removed. In cases where epoxy is used, the slot must dry prior to bonding and if cement grout is used the slot must be pre-wetted before the grout is applied. The most important factor when NSM is used is the distance to the original steel reinforcement, otherwise the pre-treatment is quite easy and the method is relatively non-sensitive to irregularities of the concrete surface.

In general the force transfer from the concrete to the strengthening component is superior for NSM systems compared to laminate and sheet systems

G.3 BENDING

G.3.1 General

FRP for strengthening are commonly used to increase the flexural capacity of concrete members. Strengthening can be done with laminates, sheets of NSM reinforcement. In general laminates are most suitable for flat surfaces such as slabs, beams and walls. Sheets are used when greater flexibility is needed, e.g. curved surfaces, columns etc. In this section a brief presentation in design for bending is presented.

G.3.2 Design for bending

Calculation in SLS

In SLS a calculation regarding stresses and strains due to service load is carried out. Here a calculation is made to investigate if the structure is cracked or not and also calculations of existing strain fields is carried out. Cross section data is then needed in the following calculations when the strain field at time for strengthening is calculated. In SLS calculations regarding deflections and crack widths may also be carried out.

Estimation of material consumption

Before a detailed design calculation is carried out it is recommended that a estimation either of the bending capacity from an material assumption or the material needed from the bending capacity needed. The bending moment capacity can be calculated according to:

$$M_d \approx 0.9\left(A_s f_y d + A_f \varepsilon_f E_f h\right) \qquad \text{(G.1)}$$

and alternatively the sectional area can be calculated according to:

$$A_f \approx \frac{\left(M_d / 0.9 - A_s f_y d\right)}{\varepsilon_f E_f h} \qquad \text{(G.2)}$$

From this a relatively good picture of the cost of the strengthening system can be generated. However, in the final design it is always recommended to carry out the calculations and steps in ULS.

Design for strengthening in ULS

In this paper only the design for single reinforced cross-sections is presented. How to design for double reinforced cross sections can be found in Täljsten (2006). In Figure G.3.1 a cross section of a rectangular strengthened concrete beam is shown. Where ε_{u0} is actual strain in the bottom fibre, ε_{c0} and ε_{s0}, strain in concrete respectively steel at time for strengthening. ε_f is the strain the FRP at ULS (or at level of calculation). $\Delta\varepsilon$ refers to the additional contribution from time of strengthening to ULS. In agreement with normal concrete design (EC2) the compressive strain in concrete, ε_c, should not exceed *3.5 ‰* in ULS. In the analysis (to calculate the FRP area) the expression in equation (G.3) is suggested. Where A_f and A_s is the cross sectional area of FRP and reinforcing steel respectively, M_d, moment capacity needed, f_y, yield stress of steel and E_f, Young´s modulus of FRP. $\lambda = 0.8$ for $f_{ck} \leq 50$ MPa.

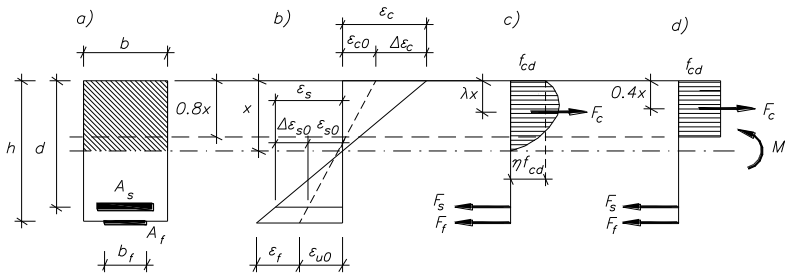


Figure G.3.1 – Single reinforced cross section

$$A_f = \frac{M_d - A_s f_y \left(d - \frac{\lambda}{2} x \right)}{\varepsilon_f E_f \left(h - \frac{\lambda}{2} x \right)} \qquad \text{(G.3)}$$

To avoid brittle compressive failures the following must be fulfilled:

$$\omega \leq \omega_{bal} \qquad \text{(G.4)}$$

where

$$\omega_{bal} = \frac{\lambda}{1 + \frac{\varepsilon_f + \varepsilon_{u0}}{\varepsilon_{cu}}} \qquad \text{(G.5)}$$

$$\omega = \frac{A_s f_y + A_f \varepsilon_f E_f}{b h f_{cd}} \qquad \text{(G.6)}$$

In cases when $\omega > \omega_{bal}$ a more accurate procedure must be followed, see Täljsten (2006). In design for strengthening in ULS different possible failure modes have to be checked. In strengthening for flexure, 7 failure modes are identified, see Figure G.3.2. These are:

1. Concrete compressive failure
2. Yielding in the tensile reinforcement (not necessary a failure mode)
3. Yielding in the compressive reinforcement (not necessary a failure mode)
4. Tensile failure in the FRP
5. Intermediate crack debonding
6. Peeling failure at the end of the laminate
7. Anchorage failure

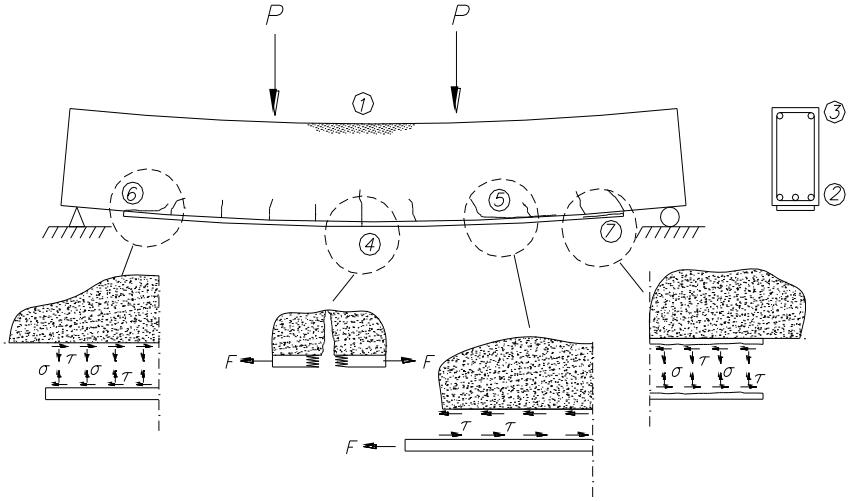


Figure G.3.2 – Possible failure modes when strengthen for flexure.

In this paper we will further discuss Intermediate crack debonding, peeling failure and anchorage failure.

In the design often IC (Intermediate Crack) debonding is governed for laminates. For NSM often strain failure in the FRP or crushing of concrete is governing. In the design guideline ACI-4402R-08 (2008) forms the basis for IC-debonding calculation, equation (G.7). However, also in this field considerably amount of research have been carried out, Smith & Teng (2002) and Toutanji et al (2007).

$$\varepsilon_{fd,ic} = 0.41\sqrt{\frac{f_{cd}}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \qquad \text{(G.7)}$$

Where f_{cd} is design compressive strength of concrete and n number of layers of FRP. The peeling failure is related to the fact that the laminate normally not can be anchored beyond the zero moment point. For a simple supported beam there will likely be a distance between the support and the plate end. At this cut-off end shear and normal stresses occur, these stresses interact and want to “peel” off the laminate from the structure, see Täljsten (1994) for a detailed analysis and Täljsten et al (2011) for a design procedure.

Proper anchoring the laminate is essential and in general the laminate shall be anchored beyond a zero moment point if possible, otherwise a laminate shall be anchored outside a cracked area to avoid IC-debonding. The anchor length is governed by the stiffness of the plate and the quality of the concrete. If debonding occurs, failure will most likely be in the concrete transfer zone. It is suggested that the anchor length is calculated with equation (G.8), where l_{ef} is effective anchor length and f_{ctm} tensile strength of concrete defined in (G.9). However, it is also recommended that, when possible, not shorter anchor length than 250 mm should be chosen.

$$l_{ef} = \sqrt{\frac{E_f t_f}{2 f_{ctm}}} \qquad \text{(G.8)}$$

where

$$f_{ctm} = 0.3 \cdot \sqrt[3]{f_{ck}^{\,2}} \quad f_{ck} = f_{cm} - 8 \text{ [MPa]} \qquad \text{(G.9)}$$

The force that should be anchored in the laminate is divided on the existing tensile steel reinforcement and the laminate, see Täljsten et al (2011). The force in the laminate in the section to be anchored is not allowed to exceed:

$$F_{f,e} = \varepsilon_{f,x} A_f E_f \qquad \text{(G.10)}$$

where

$$\varepsilon_{f,x} \leq \sqrt{\frac{2G_f}{E_f t_f}} \qquad \text{(G.11)}$$

and

$$G_f = 0.03 k_b \sqrt{f_{ck} f_{ctm}} \qquad \text{(G.12)}$$

$$k_b = \sqrt{\frac{2 - b_f / b}{1 + b_f / b}} \geq 1.0 \qquad \text{(G.13)}$$

Where k_b is a form factor that describe the size of the strengthened surface in relation to the unstrengthen surface. b_f is the width of the laminate and b the width of the structural member. Notice that the ratio $b_f/b \geq 0.33$ must be fulfilled (if $b_f/b < 0.33$ then the value of k_b should be $k_b = b_f/b = 0.33$).

G.4 SHEAR

G.4.1 General

The cause of shear failure is a result of a complicated mechanism. A combination of the effect of shearing together with shear force influence creates a multi-axial stress-state in the beam, where the maximum tensile stresses are generated at angles between *30 – 60°* (depending on reinforcement and loading) in relation to the construction's longitudinal axis. This leads to the formation of inclined shear cracks and ultimately to failure, see Figure G.4.1.

We must have a basic understanding of the behaviour and different types of shear failures to be able to strengthen concrete structures in shear. In normal situations, a concrete structure is designed to reach large deformations before failure, which means that the failure is often a bending failure. For a concrete beam with conventional steel stirrups the shear failure can be characterised in the following main categories:

- Web shear failure. Arise in those regions where the beam is not affected by bending cracks. The failure occurs when the principal tensile stress exceeds the concrete's tensile strength in the web. The failure is often a result of insufficiently or lack of shear reinforcement.
-

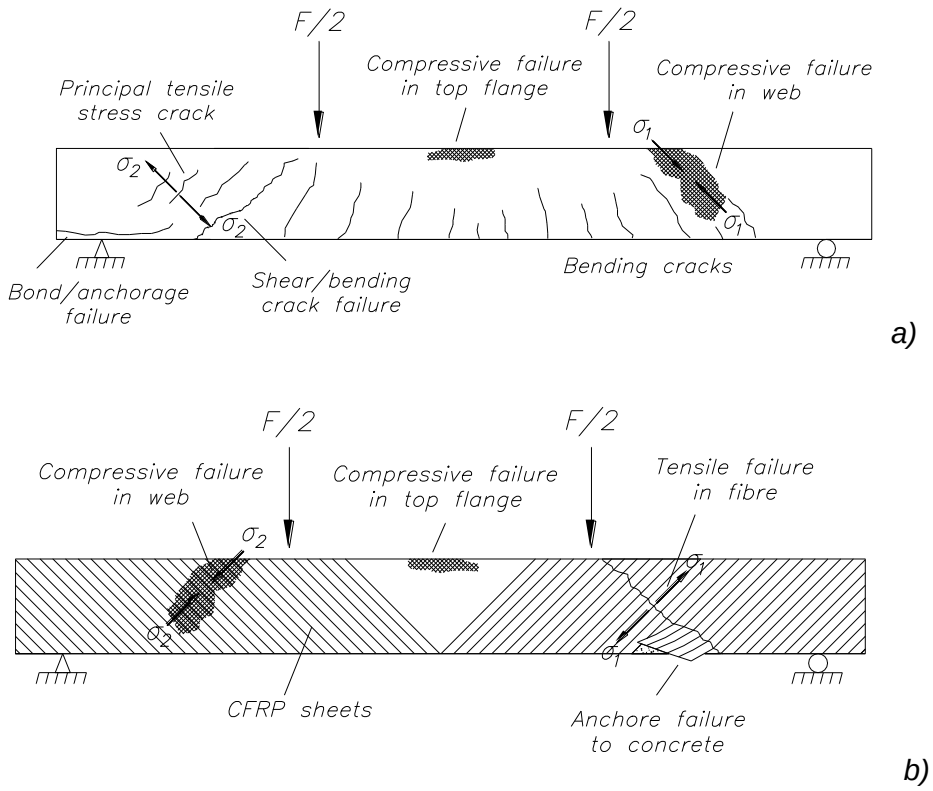


Figure G.4.1 – a) Shear failure in an unstrengthen concrete beam, b) Possible failure modes in an FRP strengthen beam

- Bending shear failure. The failure initiate from bending cracks to inclined shear cracks. The crack grows from the structure's tensile zone towards the compression zone. The final failure is crushing or splitting of the compressed zone. Shear reinforcement and external strengthening that crosses the cracked zone contributes to the shear force resistance. The shear and bending reinforcement acts as tensile bars and the concrete in the beam's compression zone and the inclined concrete struts between the shear cracks act as compressive bars in the truss model.
- Compressive failure in web. The failure is caused is caused by compression failure in the inclined concrete struts in the truss. The failure can occur when the shear reinforcement is over-dimensioned. In this case, the steel reinforcement do not reach the yield limit before the concrete's compression strength is reached.

When a concrete beam is also strengthened with external composites, another two failure modes can occur, see Figure G.4b).

- Fibre failure in the composite. Occurs when the fibre's critical strain capacity is exceeded. The failure is often characterised by a propagating failure where the composite gradually fails, especially for fabrics. The failure is usually brittle. However, the orientation of fibres in relation to the greatest principal strain affects the ductility.
- Anchorage failure. Occur when the concrete's external strength is too low or the anchorage area is too small to transfer the shear forces between the reinforcement and the concrete. In many cases this type of failure can be avoided by wrapping the beam with fabric to create closed FRP stirrups.

When strengthen concrete structures for shear is essential to anchor the strengthening material properly. Preferably is to enclose, W-wrap, a structure with a FRP sheet system. However, this is not always possible, e.g. for T-sections which is a common element to be strengthened. Here U-wrapped systems is recommended and if the end to the flange is mechanically anchored or if sufficient anchor length can be provided this is as effective as a W-wrap. Side wrapped, S-Wrap, is not recommended. Most common FRP systems for shear strengthening is sheet systems, but also laminates or NSM can and have been used.

G.4.2 Strengthening for shear

In the design suggestion below it is suggested that Eurocode, EC2, EN1992-1-1 (2004) is used. By then adding a component $V_{rd,f}$ the contribution from the FRP can be calculated. To total load carrying capacity in shear is V_{Rd}., see chapter 6.2.2 in EC2.. In the design model, see Figure G.4.2, regarding strengthening for shear with FRP consider the contribution from concrete and steel. Calculation of contribution from steel and concrete follows EC2.

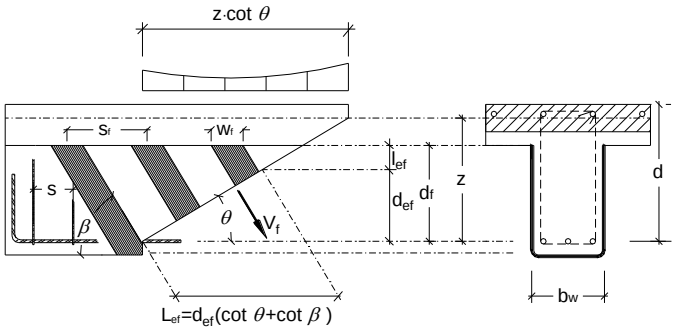


Figure G.4.2 – Design model for FRP strengthening for shear

For a beam without shear reinforcement, but with FRP strengthening the capacity is:

$$V_{Rd} \le V_{Rd,f} \qquad \text{(G.14)}$$

and for a beam with shear reinforcement and with FRP strengthening the load carrying capacity is calculated by:

$$V_{Rd} \le min\left(V_{Rd,s} + V_{Rd,f}\,; V_{Rd,max}\right) \qquad \text{(G.15)}$$

where $V_{Rd.s}$ and $V_{Rd,max}$ is decided due to section 6.2.3 in EC2.. The contribution to the shear capacity can be calculated in (G.16) where the strain level is limited, see Täljsten et al (2011).

$$V_{Rd,f} = A_f \varepsilon_{fd} E_{fd} L_{ef} \sin\beta_f \cos^2\alpha \qquad \text{(G.16)}$$

where the anchor length l_{ef}, is calculated in Eqn. (6.8). The effective length, L_{ef}, is calculated as:

$$L_{ef} = d_{ef} \cdot \left(\cot\theta + \cot\beta_f\right) \qquad \text{(G.17)}$$

and

$$d_{ef} = \begin{cases} z = 0.9d & for\ W-wrap \\ min\left(z; d_f - l_{ef}\right) & for\ U-wrap \end{cases} \qquad \text{(G.18)}$$

G.5 CONFINEMENT

If a column need to be strengthened and if the dimensions are to be kept the most beneficial method to increase the load bearing capacity of an existing column is to apply a confinement pressure. It has been proven by numerous researchers that a confinement pressure can enhance the load bearing capacity of axial loaded member, Richart et al (1928), Mander et al. (1988). In a traditional reinforced structural member, the confinement is provided for by lateral steel reinforcement. The lateral steel induces compressive confining stresses on the concrete core, due to the elongation of the steel, which is caused by the expansion of the concrete, the Poisson effect. When the axial strain increases, the confining pressure in the two transverse directions increases, and the strength of the concrete core in the axial direction is enhanced as well. The confinement of concrete columns is a well-established technique for improving both compressive behaviour and flexural response. Traditionally for new built columns, this is taken care of in design of the steel stirrups. However, for repair and strengthening of existing columns this can be provided by wrapping by FRP. Depending on the purpose of the repair or strengthening scheme the shear strength, axial strength and/or ductility can be enhanced. Using FRP for strengthening of columns has shown to be very efficient. The confinement effect is used in the calculation to create an increased compressive strength in the concrete. For uniaxial loaded columns this calculation is very straight forward and easy to carry out. However, in the presented guideline also the effect of a bending moment is considered. The dominating strengthening system for confinement is sheet systems where FRP sheets are wrapped around a column. The system is most effective for circular columns but can also be used for rectangular columns with some reduction factors. The maximum increased compressive strength for a wrapped concrete column can be calculated as:

$$f_{cd,c} = f_{cd} + \alpha_{f,c} 3.3 \kappa_a f_l \qquad \text{(G.19)}$$

and

$$f_l = \frac{2E_f n t_f \varepsilon_{fe}}{D} \qquad \text{(G.20)}$$

where κ_a is an efficiency factor that consider the geometry of the cross section (circular or not). $\alpha_{f,c}$ = 0.95, saftey factor. And the effective strain is given by:

$$\varepsilon_{fe} = 0.004 \leq \kappa_\varepsilon \varepsilon_{fu} \qquad \text{(G.21)}$$

κ_e is an efficiency factor considering possible premature failure regarding to tri-axial state of stress for wrapped sections.

The design for confinement when a combined normal force and a bending moment act on a axial member is best explained by figure G.5.1 a) and G.5.1 b), see Täljsten et al (2011). Here we calculate different stages for a rectangular member, A to D, before and after strengthening. In figure G5.1b we can notice the capacity before strengthening – the dashed curve, the need for strengthening, the *X* and the capacity after strengthening, the solid line.

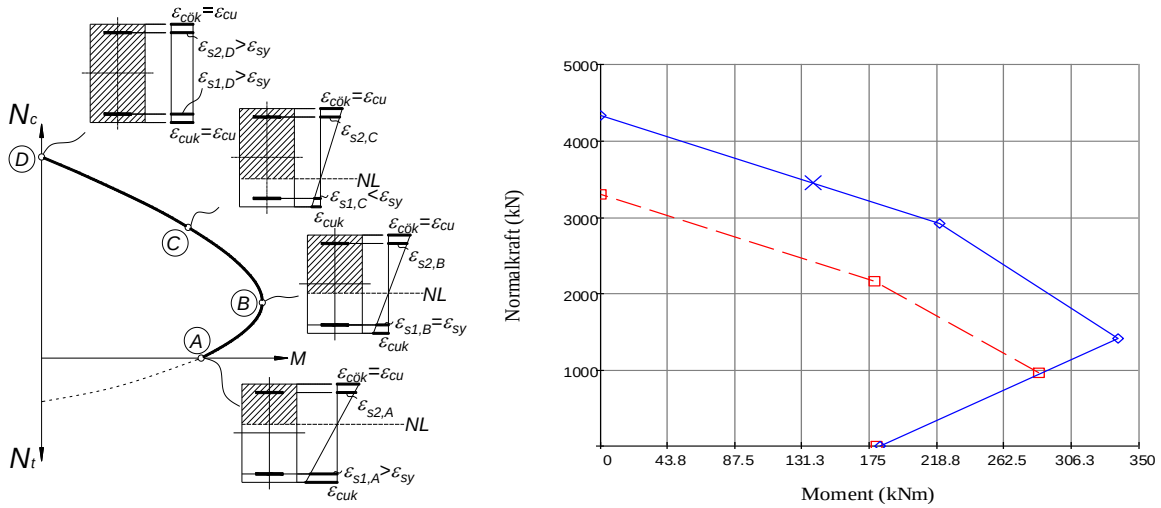


Figure G.5.1 Strengthening for confinement. a) General calculation, b) calculation for strengthening

Appendix H – Design Examples

H.1 - Design Example Bending

The beam in Figure H.1 is a part of a floor structure in a parking garage and needs to be strengthened for additional load. The general procedure will be the same for a railway bridge. The beam is for simplicity assumed to be freely supported and have a distributed load over the whole length, 8 m, of the beam. The beam is loaded with a sagging moment of 200 kNm in SLS and additional loading is calculated to 430 kNm in the ULS. The creep number is 2.0 and the geometrical conditions can be found in Figure H1.1 and Table H1.2. The strengthening shall be carried out with CRFP plates. The material properties are shown in Table H1.3. During strengthening the loading can be reduced to 170 kNm.

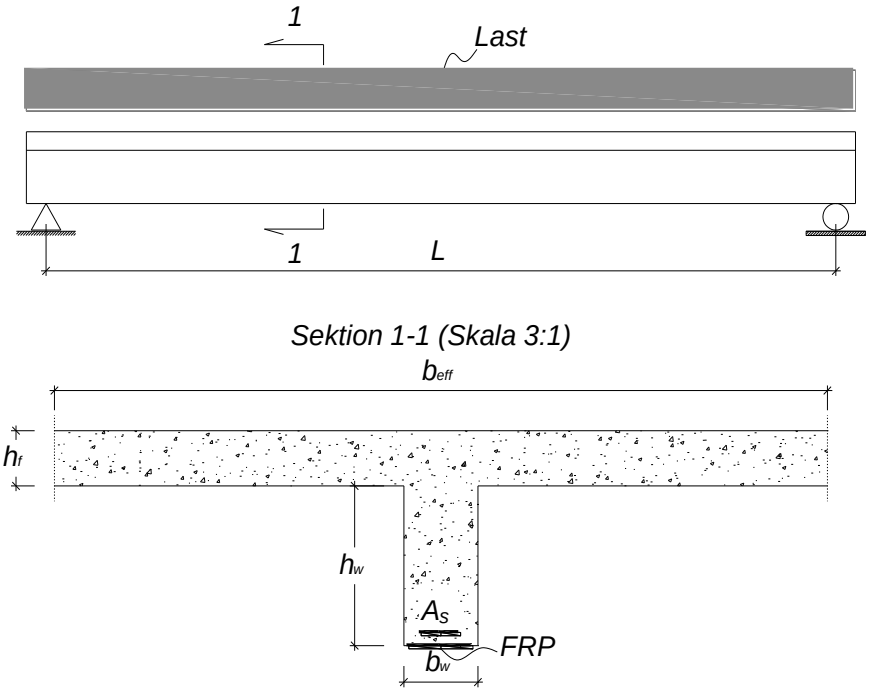


Figure H1,1 Geometry for structure to be strengthened

The caculation for concrete follows Eurocode 2. The partical coeffecients (safety factors) is shown in Table H1.1

Table H1.1 Partical coefficients and parameters in ULS.

Concrete	Steel	FRP
γ_c =1.5	γ_s =1.15	γ_{frp} =1.2
α_{cc}=0.85	α_{ct}=0.85	
φ_{ef}=2.0		
γ_{cE} =1.2		

Table H1.2 Geometrical conditions for beam and steel reinforcement

Notation	Value	Unit	Description
$b_f = b_{eff}$=	2610	mm	Effective flange width(EC2 5.3.2.1)
h_f=	180	mm	Height on flange
h_w=	520	mm	Height on web
h=	700	mm	Total height
c=	30	mm	Concrete cover
b_w=	250	mm	Width web
A_c=	599800	mm2	Cross sectional area concrete
A_s=	1256.6	mm2	Area tensile reinforcement
$Ø_t$=	20	mm	Diameter steel reinforcement
d=	660	mm	Level arm
L=	8000	mm	Distance between supports
B =	5000	mm	Distance between beams
A_{sw}=	157.1	mm2	Area shear reinforcement
$Ø_s$=	10	mm	Diameter shear reinforcement
s=	250	mm	Internal distance shear reinforcement

Step 1. Investigate the existing stage

It is important to investigate the initial condition for the structure. Start by calculating design values for the materials used due to Eurucode 2 and Täljsten et. al. (2011). These values are shown in table H1.3.

Table H1.3 Characteristic and design values for reinforcing steel and concrete

Concrete	Characteristic values			Steel	Characteristic values		
	f_{ck}	40	MPa		f_{yk}	500	MPa
	f_{ctm}	3.5	MPa		E_s	210	GPa
	E_{cm}	35	GPa				
Concrete	Design values			Steel	Design values		
	f_{cd}	22.6	MPa		f_{yd}	435	MPa
	f_{ctm}	3.5	MPa		E_{sd}	183	GPa

Note that the Modulus of Elasticity for concrete is given in the Ultimate limit State (ULS) and the Serviceability Limit State (SLS). Start to calculate the proportionality factor between steel and concrete in SLS:

$$\alpha_s = \frac{E_{sd}}{E_{c,eff}} = \frac{E_s\left(1+\varphi_{ef}\right)}{E_{cm}} = \frac{210(1+2)}{35} = 18.0$$

Then check the beam with the original loads in the SLS. Start to calculate the distance to the neutral axis for the combined cross section:

$$y_o = \frac{b_{eff}h_f\frac{h_f}{2} + b_w h_w\left(h_f + \frac{h_w}{2}\right) + \left(\alpha_s - 1\right)A_s d}{b_f h_f + b_w h_w + \left(\alpha_s - 1\right)A_s} =$$

$$= \frac{2610\cdot 180\cdot\frac{180}{2} + 250\cdot 520\left(180 + \frac{520}{2}\right) + \left(18.0-1\right)\cdot 1256.6\cdot 660}{2610\cdot 180 + 250\cdot 520 + \left(18.0-1\right)1256.6} =$$

$$= 182.9mm$$

The moment of gravity can then be calculated as:

$$I_1 = I_c + \left(\alpha_s - 1\right)I_s = \frac{b_f h_f^3}{12} + b_f h_f\left(y_0 - \frac{h_f}{2}\right)^2 + \frac{b_w h_w^3}{12} + b_w h_w\left(y_0 - h_f - \frac{h_w}{2}\right)^2 +$$

$$+\left(\alpha_s - 1\right)A_s\left(d - y_0\right)^2 =$$

$$= \frac{2610\cdot 180^3}{12} + 2610\cdot 180\left(182.9 - \frac{180}{2}\right)^2 + \frac{250\cdot 520^3}{12} +$$

$$+250\cdot 520\left(182.9 - 180 - \frac{520}{2}\right)^2 + \left(18.0-1\right)\cdot 1256.6\cdot\left(660 - 182.9\right)^2 =$$

$$= 2.17\cdot 10^{10}\, mm^4$$

Then we calculate the maximum stress in the reinforcement and in the most tensioned part of the beam:

$$\sigma_s = \frac{M_0}{I_c + \left(\alpha_s - 1\right)I_s}\left(d - y_0\right) = \frac{M_0}{I_1}\left(d - y_0\right) =$$

$$= \frac{200\cdot 10^6}{2.17\cdot 10^{10}}\left(660 - 182.9\right) = 4.40\,MPa$$

$$\sigma_{cu} = \frac{M_0}{I_c + \left(\alpha_s - 1\right)I_s}\left(h - y_0\right) = \frac{M_0}{I_1}\left(h - y_0\right) =$$

$$= \frac{200\cdot 10^6}{2.17\cdot 10^{10}}\left(700 - 182.9\right) = 4.76\,MPa$$

According to Eurocode 2 the cross section is cracked if the bending stress is higher than f_{ctm}. Here σ_{cu} = 4.76 > f_{ctm}. = 3.5 MPa. The moment of gravity will now be calculated for stage II. We assume that neutral axis will be located in the flange and obtain the following equilibrium:

$$\frac{b_f x^2}{2} = \alpha_s A_s\left(d - x\right) \Rightarrow \underbrace{\frac{b_f}{2}}_{A}x^2 + \underbrace{\alpha_s A_s}_{B}x + \underbrace{\left(-\alpha_s A_s d\right)}_{C}$$

Here the distance *x* can be found from this second degree equation:

$$x = -\frac{B}{2A} \pm \sqrt{\left(\frac{B}{2A}\right)^2 - \frac{C}{A}} = \frac{25581}{2\cdot 1305} \pm \sqrt{\left(\frac{25581}{2\cdot 1305}\right)^2 - \frac{17061541}{1305}} = 104.5mm$$

The assumption that the neutral axis is situated in the flange was correct, so the moment of gravity for stage II can now be calculated

$$I_2 = I_c + \alpha_s I_s = \frac{b_f x^3}{12} + b_f x\left(\frac{x}{2}\right)^2 + (\alpha_s - 1)A_s(d-x)^2 =$$

$$= \frac{2610 \cdot 104.5^3}{12} + 2610 \cdot 104.5\left(\frac{104.5}{2}\right)^2 +$$

$$+(20.6-1)1256.6(660-104.5)^2 = 0.90 \cdot 10^{10}\, mm^4$$

Step 2. Determine the initial strains and stresses

We assume that plane sections remain plane and calculate the stresses in the concrete and the reinforcement for the moment M_{01} = 200 kNm in SLS

$$\sigma_{cö} = \frac{M_{01}}{I_2}x = \frac{200 \cdot 10^6}{0.90 \cdot 10^{10}}104.5 = 2.33\, MPa$$

$$\sigma_s = \alpha_s \frac{M_{01}}{I_2}(d-x) = 20.6\frac{200 \cdot 10^6}{0.90 \cdot 10^{10}}(660-104.5) = 254.80\, MPa$$

Both stresses are lower than the design limits. The stresses during the strengthening are obtained in the same way for M_{02} = 170 kNm

$$\sigma_{cö} = \frac{M_{02}}{I_2}x = \frac{170 \cdot 10^6}{0.90 \cdot 10^{10}}104.5 = 1.98\, MPa$$

$$\sigma_s = \alpha_s \frac{M_{02}}{I_2}(d-x) = 20.6\frac{170 \cdot 10^6}{0.90 \cdot 10^{10}}(660-104.5) = 216.58\, MPa$$

The corresponding strains are:

$$\varepsilon_{cö} = \varepsilon_{c0} = \frac{\sigma_{cö}}{E_{eff}} = \frac{1.98}{11.67 \cdot 10^3} = 0.17 \text{ ‰}$$

$$\varepsilon_s = \varepsilon_{s0} = \frac{\sigma_s}{E_{sd}} = \frac{216.58}{183 \cdot 10^3} = 1.18 \text{ ‰}$$

$$\varepsilon_u = \varepsilon_{s,M_{02}} = \frac{\varepsilon_s(h-x)}{d-x} = \frac{1.08(700-104.5)}{660-104.5} = 1.16 \text{ ‰}$$

Step 3. Estimate the required strengthening

Start to estimate the required strengthening by using the simplified formula. Characteristic and design values are given in Table H1.4

Tabel H1.4 Characteristic and Design Values for FRP

FRP	Characteristic values	Design values

ε_{fk}	15	‰	ε_f	12.50	‰
E_{fk}	160	GPa	E_f	133.33	GPa

The design strain must be limited due to the risk for intermediate cracking from Eq. (G.7)

$$\varepsilon_{fd,ic} = 0.41\sqrt{\frac{f_{cd}}{nE_{fd}t_f}} = 0.41\sqrt{\frac{22.67}{1\cdot 133.33\cdot 10^3\cdot 1.4}} = 4.52 \text{ ‰}$$

Now the simplified design formula can be used

$$A_f = \frac{M_d/0.9 - A_s f_{yd} d}{\varepsilon_f E_f h} = \frac{430\cdot 10^6/0.9 - 1256.6\cdot 434.78\cdot 660}{4.52\cdot 10^{-3}\cdot 133.33\cdot 10^3\cdot 700} = 277mm^2$$

The estimated area corresponds to two laminates 100 x 1.4. Now when the area is estimated, we can calculate the depth of the compression zone. We neglect shrinkage due to the fact that the shrinkage mostly is completed at the time of strengthening. We use the following equilibrium equation:

$$x = \frac{A_s f_{yd} + \varepsilon_{fd,ic} E_{fd} A_f}{\lambda\zeta f_{cd} b_w} = \frac{1256.6\cdot 434.78 + 4.52\cdot 133.33\cdot 280}{0.8\cdot 1.0\cdot 22.67\cdot 250} = 157.7mm$$

Now the moment capacity can be calculated from the following equilibrium:

$$M = A_s f_{yd}\left(d - \frac{\lambda}{2}x\right) + \varepsilon_{fd,ic} E_{fd} A_f\left(h - \frac{\lambda}{2}x\right) =$$

$$= 1256.6\cdot 434.78\left(660 - \frac{0.8}{2}157.7\right) + 4.52\cdot 133.33\cdot 280\left(700 - \frac{0.8}{2}157.7\right) =$$

$$= 433.6kNm$$

The moment capacity of the strengthened section is higher than the required moment M_d = 430 kNm.

Step 4. Check that the reinforcement yields (normally reinforced section)

It is important to check the failure mode and that the concrete does not fail in compression. We check the strains and stress in a balanced section and for a section with compression failure

$$\omega_{bal} = \frac{0.8}{1 + \frac{\varepsilon_{fd,ic} + \varepsilon_{u0}}{\varepsilon_{cu}}} = \frac{0.8}{1 + \frac{4.52 + 1.16}{3.5}} = 0.305$$

A cross section with maximum reinforcement is obtained for

$$\omega = \frac{A_s f_{yd} + A_f \varepsilon_{fd,ic} E_{fd}}{b_{eff} h f_{cd}} = \frac{1256.6\cdot 434.78 + 280\cdot 4.52\cdot 133.33}{2610\cdot 700\cdot 22.67} = 0.017$$

If $\omega_{bal} > \omega$ the section is normally reinforced

Step 5.Design of required bond length

(a) Calculate the distance to the last crack, x_{cr}, where the bending capacity corresponds to the cracking moment. A simplified calculation, on the safe side, is to use the bending capacity without reinforcement.

Calculate the center of gravity for the cross section:

$$y_0 = \frac{b_{eff} h_f \frac{h_f}{2} + b_w h_w \left(h_f + \frac{h_w}{2} \right)}{b_{eff} h_f + b_w h_w} =$$

$$= \frac{2610 \cdot 180 \cdot \frac{180}{2} + 250 \cdot 520 \left(180 + \frac{520}{2} \right)}{2610 \cdot 180 + 250 \cdot 520} = 165.9 mm$$

Calculate the moment of gravity:

$$I_c = \frac{b_{eff} h_f^3}{12} + b_{eff} h_f \left(y_0 - \frac{h_f}{2} \right)^2 + \frac{b_w h_w^3}{12} + b_w h_w \left(y_0 - h_f - \frac{h_w}{2} \right)^2 =$$

$$= \frac{2610 \cdot 180^3}{12} + 2610 \cdot 180 \left(165.9 - \frac{180}{2} \right)^2 + \frac{250 \cdot 520^3}{12} +$$

$$+ 250 \cdot 520 \left(165.9 - 180 - \frac{520}{2} \right)^2 =$$

$$= 1.67 \cdot 10^{10} mm^4$$

Calculate the bending resistance:

$$W_c = \frac{I_c}{y_0} = \frac{1.67 \cdot 10^{10}}{165.9} = 1.01 \cdot 10^8 mm^3$$

Calculate the moment when the last crack appears:

$$M_{x_{cr}} = W_c f_{ctm} = 1.01 \cdot 10^8 \cdot 3.5 = 351.80 kNm$$

Calculate the distance to the last crack. We assume the load to be evenly distributed without any point loads. We also assume the beam to be simply supported. The moment and shear equations can then be written

$$M_x(x) = R_A x - q \frac{x^2}{2}$$

$$V_x(x) = R_A - qx$$

The support reaction is

$$R_A = \frac{qL}{2} = \frac{53.8 \cdot 8 \cdot 10^3}{2} = 215 kN$$

With the load

$$\frac{q}{2} = \frac{53.8}{2} = 26.9 kN / m$$

By solving x from the moment equation we get the distance to the last crack for $M_{x_{cr}}$

$x = 2294.2mm$

(b) Calculate the distance a_l and the moment M_{x_a} in section x_a, see Figure H.2

Here we use a simplified expression for a crack angle of 45° with stirrups at 90°

$$a_l = 0.45d = 0.45 \cdot 660 = 297.0mm$$

The moment M_{x_a} is relocated the distance a_l from the point for the last crack x_{cr},.and can be obtained from the moment equation:

$$M_{x_a} = 376.66kNm$$

(c) Calculate the tensile force in the composite which carries the moment M_{x_a} together wirth the steel reinforcement

As the steel reinforcement yields at failure we obtain:

$$F_s = A_s f_{yd} = 1256.6 \cdot 434.78 = 546.4kN$$

The force in the composite will be:

$$F_f = \frac{M_{x_a}}{0.9h} - F_s \frac{d}{h} = \frac{376.66}{0.9 \cdot 700} - 546.36 \cdot 10^3 \frac{660}{700} = 82.73kN$$

or

$$F_f = \frac{M_{x_a}/0.9h}{1 + \frac{E_{sd}A_s}{E_{fd}A_f}\left(\frac{d}{h}\right)^2} = \frac{\frac{376.66}{0.9 \cdot 700}}{1 + \frac{200 \cdot 10^3 \cdot 1256.6}{133.33 \cdot 10^3 \cdot 280}\left(\frac{660}{700}\right)^2} = 85.60kN$$

Choose the value which gives the highest value F_f = 85.60*kN*

(d) Check that the calculated force $F_f \leq F_{f,e}$, the allowed effective force to be able to anchor the composite to the concrete

$F_{f,e} = \varepsilon_{f,x} A_f E_{fd}$ where

$$k_b = \sqrt{\frac{2 - b_f/b_c}{1 + b_f/b_c}} = \sqrt{\frac{2 - \frac{2 \cdot 100}{250}}{1 + \frac{2 \cdot 100}{250}}} = 0.82 \quad \text{(shall be ≥1)}$$

$$G_f = 0.03k_b\sqrt{f_{ck}f_{ctm}} = 0.03 \cdot 1.0\sqrt{40 \cdot 3.5} = 0.35Nmm/mm^2$$

$$\varepsilon_{f,x} \leq \sqrt{\frac{2G_f}{E_{fd}t_f}} = \sqrt{\frac{2 \cdot 0.35}{133.33 \cdot 10^3 \cdot 1.4}} = 1.95$$

The strain in the composite at the force F_f replaced from the first crack will then be

$$\varepsilon_f = \frac{F_f}{E_{fd}A_f} = \frac{72.81 \cdot 10^3}{133.33 \cdot 10^3 \cdot 280} = 2.29 \text{ ‰}$$

As the strain in the composite is larger than the allowed strain, $\varepsilon_{f,x} < \varepsilon_f$, a new value for the anchorage must be calculated. The allowed force in the composite will be

$$F_{f,e} = \varepsilon_{f,x} A_f E_{fd} = 1.95 \cdot 280 \cdot 133.33 = 72.81 kN$$

By solving for F_f we can obtain the moment at anchorage

$$M_{f,e} = 0.9 h F_{f,e} \left(1 + \frac{E_{sd} A_s}{E_{fd} A_f} \left(\frac{d}{h} \right)^2 \right) =$$

$$= 0.9 \cdot 700 \cdot 72.81 \cdot 10^3 \left(1 + \frac{200 \cdot 10^3 \cdot 1256.6}{133.33 \cdot 10^3 \cdot 280} \left(\frac{660}{700} \right)^2 \right) = 320.37 kNm$$

and

$$M_{f,e} = 0.9 h \left(F_{f,e} + F_s \left(\frac{d}{h} \right) \right) =$$

$$= 0.9 \cdot 700 \left(72.81 \cdot 10^3 + 546.36 \cdot 10^3 \left(\frac{660}{700} \right) \right) = 370.41 kNm$$

Chose the smallest from the two moments:

$M_{f,e}$ = 320.37*kNm*

The distance where the anchorage shall start, $x_{f,e}$ can be calculated from the moment equation

$$M_{x_{f,e}} = R_A x_{f,e} - \frac{q x^2}{2} \Rightarrow x_{f,e} = 1980.3 mm$$

We check if the required anchorage length is reached, Eq. (G.8):

$$l_e = \sqrt{\frac{E_{fd} t_f}{2 f_{ctm}}} = \sqrt{\frac{133.33 \cdot 10^3 \cdot 1.4}{2 \cdot 3.5}} = 163.3 mm$$

The length should be chosen to be at least 250 mm. Depending on how the situation is at the support, i.e. how much free distance a that is available, a solution as in figure H1.2 can be obtained.

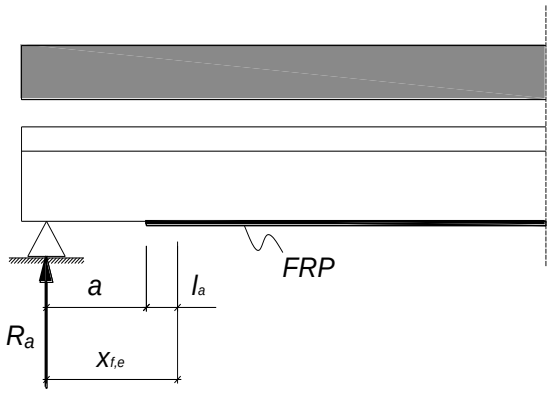


Figure H1.2. Anchorge length

$$a = x_{f,e} - l_e = 1980.3 - 250 = 1730.3 mm$$

As the length *a* is relatively large it is possible to anchor the force closer to the support in order to obtain a more favourable stress condition. In this case it is suggested to anchor the composite as close to the support as possible.

Step 6. Check splitting forces at the end of the composite

(e) Choose a distance from the support to the end of the composite

a = 100 mm is chosen (by anchoring the composite as close to the support as possible, the shear stresses are minimised).

The maximum shear stress for a distributed load will the be

$$\tau_{\max} = \frac{q}{2}\frac{G_a}{sE_{cd}W_c}\frac{\left(a^2+2al\right)\lambda+l}{\lambda^2} =$$

$$= 26.9\frac{4.7\cdot 10^3}{2\cdot 29.17\cdot 10^3\cdot 1.00\cdot 10^8}\frac{\left(100^2+2\cdot 100\cdot 4000\right)0.109+4000}{0.109^2} = 0.16MPa$$

Here $l = L/2$ and $z_0 = h - x$ and

$$\lambda = \sqrt{\frac{G_a b_f}{s}\left[\frac{1}{E_{fd}A_f}+\frac{1}{E_{cd}A_c}+\frac{z_0}{E_{cd}W_c}\right]} =$$

$$= \sqrt{\frac{4.7\cdot 10^3\cdot 2\cdot 100}{2}\left[\begin{array}{l}\frac{1}{133.33\cdot 10^3\cdot 280}+\frac{1}{29.17\cdot 10^3\cdot 599800}+\\ +\frac{595.52}{29.17\cdot 10^3\cdot 1.00\cdot 10^8}\end{array}\right]} = 0.109$$

(f) Calcualte the tensile force in the bottom of the cross section at the end of the laminate. The cross section can be regarded as without cracks as $a << x_{cr}$. Note that the shear and splitting stresses only are influenced by the new loads after strengthening.

Q_{befor} = 21.25*kN/m* (dead load from slab + existing load M_{01})

$$\Delta q = q_{efter} - q_{före} = 53.75 - 21.25 = 32.5kN/m$$

This gives the support reaction, which was earlier exprressed as

$$R_A = \frac{qL}{2} = \frac{32.5\cdot 8000}{2} = 130kN$$

Calculate the moment at *a* = 100*mm*

$$M_x(x) = R_A a - q\frac{a^2}{2} = 130\cdot 10^3\cdot 100 - 32.5\cdot 10^3\frac{100^2}{2} = 12.7kNm$$

Calculate the stress at the bottom of the cross section

$$\sigma_x = \frac{M_x}{I_1}\left(h-y_0\right) = \frac{12.7\cdot 10^6}{2.17\cdot 10^{10}}\left(700-182.9\right) = 0.30MPa$$

(f) Use the following failure criterion for the principle stresses at the end of the laminate

$\sigma_1 < f_{ctm}$

The principle stress can be written:

$$\sigma_1 = \frac{\sigma_x + \sigma_y}{2} + \sqrt{\left[\left(\frac{\sigma_x - \sigma_y}{2}\right)^2 + \tau_{xy}^2\right]} =$$

$$= \frac{0.30 + 0.16}{2} + \sqrt{\left(\frac{0.30 + 0.16}{2}\right)^2 + 0.16^2} = 0.40 MPa$$

Please note that simplifying assumptions on the safe side has been used by assuming that $\sigma_y = \tau_{xy} = \tau_{max}$.

The condition for (f) is thus fulfilled and the splitting forces are within the tolerances and no mechanical anchorage is needed. If the anchorage had been chosen at *a* = 1790.8*mm,* then the splitting stresses had been much higher (σ_1 = 6.39*MPa*) and the anchorage at that point had not been possible. An optimization of the anchorage had been to anhor the laminate at *a* = 944*mm*.

H.2 - Design Example Shear

The beam in Figure H2.1 will be strengthened due to a mistake during construction. The stirrups were placed with a spacing of 250 mm instead of the design value of 150 mm. The beams were originally designed for a load of 70 kN/m, which corresponds to a vertical shear force of 280 kN and a bending moment of 560 kNm. The effective creep value can be assumed to be 2.0 and the geometrical properties are given in Figure H2.1 and Table H2.1. The strengthening will be carried out with a Carbon Fibre Textile applied vertically and with properties as in Table H2.3.

The calculations are given here as an example and they follow an iterative process. It is recommended to use a program e.g. a spread sheet to facilitate the design calcualtions. Normally convergence is obtained after some five iterations.

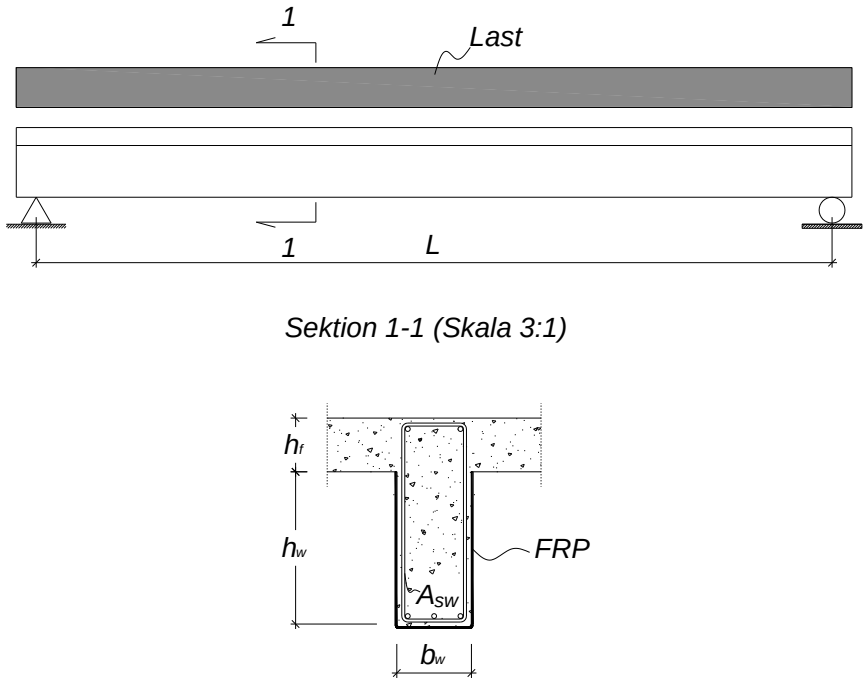


Figure H2.1 Geometry for shear example

Table H2.1 Geometry for beam and reinforcement

Notation	Value	Sort	Description
b_f=	600	mm	Effective flange width (EC2 5.3.2.1)
h_f=	150	mm	Flange height
h_w=	650	mm	Web height
b_w=	300	mm	Web width
h=	800	mm	Total height
c=	30	mm	Concrete cover
d=	755	mm	Effective height
A_{sw}=	157.08	mm²	Stirrup area
ϕ_s=	10	mm	Stirrup diameter
s=	250	mm	Stirrup spacing
L=	8	m	Beam length

The calculations are based on Eurocode 2 (EC2) and the reduction factors in Table H2.2 originate there. For FRP coefficients are proposed in Täljsten et al (2011).

Table H2. Reduction factors in the Ultimate Limit State

Concrete	*Steel*	*FRP*
$\gamma_c = 1.5$	$\gamma_s = 1.15$	$\gamma_{frp} = 1.35$
$\alpha_{cc} = 0.85$	$\alpha_{ct} = 0.85$	
$\varphi_{eff} = 2.0$		
$\gamma_{cE} = 1.2$		

Characteristic and design values are given in Table H2.3

Table H2.3. Characteristic and design values for concrete and steel

Concrete				*Steel*			
	Characteristic values				*Characteristic values*		
	f_{ck}	40	MPa		f_{yk}	500	MPa
	f_{ctm}	3.5	MPa		E_s	210	GPa
	E_{cm}	35	GPa				
	Design values				*Design values*		
	f_{cd}	22.6	MPa		f_{yd}	435	MPa
	f_{ctm}	3.5	MPa		E_{sd}	183	GPa

According to Eq. (G.15) the design value of the shear capacity is given by

$$V_{Rd} \le min\left(V_{Rd,s} + V_{Rd,f}; V_{Rd,max}\right)$$

Step 1. Determine the present shear capacity

According to EC2 the shear capacity can be determined to

$$V_{Rd,s} = \frac{A_{sw}}{s} 0.9 d f_{yd} \cot\theta = \frac{157.1}{250} \cdot 0.9 \cdot 755 \cdot 435 \cdot 1 = 186 kN$$

The upper limit is determined by the concrete capacity in the compression struts:

$$V_{Rd,\max} = \alpha_{cw} b_w 0.9 d v_1 f_{cd} / \left(\cot\theta + \tan\theta\right) =$$
$$= 1 \cdot 300 \cdot 0.9 \cdot 755 \cdot 0.6 \cdot 26.6 / \left(1+1\right) = 1630.8 kN$$

The shear capacity of the beam is the smallest value of $V_{Rd,s}$ and $V_{Rd,max}$, i.e. V_{Rd} =186 kN.

The missing capacity is then 280 – 186 = 94 kN. This load shall be carried by the shear strengthening. To estimate how much that is required the values from Table H2.4 are used

Table H2.4 Characteristic and design values for CPRP

FRP	*Characteristic values*			*Design values*		
	ε_{fk}	1.8	%	ε_f	1.33	%
	E_{fk}	228	GPa	E_f	169	GPa

Step 2- Determine the effective length L_{EF}

Determine the characteristic anchorage length by Eq (G.8). We assume that we have a carbon fibre with the thickness 0.17 mm (for a textile with a weight of 300 g/m^2)

$$l_{ef} = \sqrt{\frac{E_f t_f}{2 f_{ctm}}} = \sqrt{\frac{169 \cdot 10^3 \cdot 0.17}{2 \cdot 3.5}} = 64mm$$

Note that in Eq (G.8) the sorts *N* and *mm* are used. Calculate the available anchorage length, d_{ef}, by using Eq. (G.18)

$$d_{ef} = \min\left(z; d_f - l_{ef}\right) = \min\left(0.9 \cdot 755; d - h_{fl} - l_{ef}\right) = 540.96mm$$

Finally the effective length can be calculated with Eq. (G.17)

$$L_{ef} = d_{ef} \cdot \left(\cot\theta + \cot\beta_f\right) = 540.96 \cdot \left(\cot 45 + \cot 90\right) = 540.96mm$$

Step 3. Calculate the effective strain ε_{ef}

Use Eq. (5.11) in Täljsten et al (2011) to calculate the geometry factor

$$k_b = \sqrt{\frac{2 - w_f / s_f}{1 + w_f / s_f}} = \sqrt{\frac{2-1}{1+650/650}} = 0.707 \le 1 \quad \Rightarrow \quad k_b = 1$$

Calculate the fracture energy by using Eq. (5.10) in Täljsten et al (2011)

$$G_f = 0.03 k_b \sqrt{f_{ck} f_{ctm}} = 0.03 \cdot 1 \cdot \sqrt{22.6 \cdot 2.5} = 0.355 N\,mm/mm^2$$

Determine the effective anchorage strain by Eq (5.9) in Täljsten et al (2011):

$$\varepsilon_{fb,d} = \sqrt{\frac{2G_f}{E_f t_f}} = \sqrt{\frac{2 \cdot 0.354965}{169 \cdot 10^3 \cdot 0.17}} = 0.5\%$$

Finally the effective strain is determined to $\varepsilon_{ef} = 0.5\%$

Step 4. Determine required strengthening

By using Eq (G.16) the thickness t_f of the textiles can be determined

$$t_f = \frac{V_f}{2\varepsilon_f E_f L_{ef} \sin\beta_f} = \frac{94 \cdot 10^3 [N]}{2 \cdot 0.5 \cdot 10^{-2} \cdot 169 \cdot 10^3 \cdot 540.96 \cdot 1} = 0.208mm$$

This means that the required thickness is 0.208 mm, which is more than the estimate in Step 2 (0.17 mm). As the thickness influences the effective strain and the effective anchorage length, the calculation has to be redone from Step 2. In this example the iterative procedure will give a thickness of 0.27 mm which corresponds to two layers of textile 2·0.17 = 0.34 mm.

Step 5. Verify the shear capacity .

For the chosen thickness of 0.34 mm, the shear capacity can be determined. The capacity is calculated by using Step 2 and Step 3. To

$$V_f = 103.87kN > 94kN \qquad ok!$$

H.3 - Design Example Confinement

The column in figure H3.1 is exposed for a normal force of 3000 kN (e=0.1h) and a bending moment of 120 kNm. We want to increase the normal force with 15% to 3450 kN and the bending moment will then be 138 kNm. Design for FRP strengthening in confinement for the additional load. Geometries can be found in table H3.1 and the material parameters in Table H3.3.

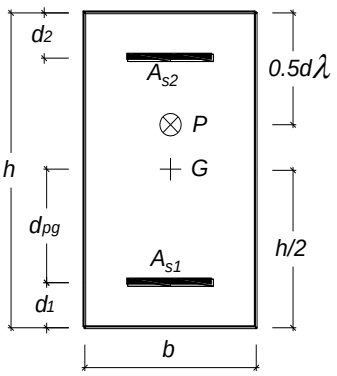


Figure H3.1. Section of column

Table H3.1. Geometries

Notation	*Numbers*	*Unit*	*Description*
$b = h$	400	*mm*	*Column width/breadth*
d_1=	40	*mm*	*Lever arm reinforcement*
d_2=	40	*mm*	*Lever arm reinforcement*
A_{s1}=	1256	mm^2	*Area reinforcement*
A_{s2}=	1256	mm^2	*Area reinforcement*
A_c=	157487	mm^2	*Area concrete, without steel*
A_g=	160000	mm^2	*Gross area*

Calculations due to Eurocode 2. Partial coefficients for the concrete, steel and FRP can be found in table C.2.

Table H3..2 Partial coefficient in ULS

Concrete	*Steel*	*FRP*
γ_c =1.5	γ_s =1.15	γ_{frp} =1.35
α_{cc}=0.85	α_{ct}=0.85	
φ_{ef}=2.0		
γ_{cE} =1.2		

Table H3.3 Characteristic and design values for steel and concrete

Concrete	*Characteristic values*			Steel	*Characteristic values*		
	f_{ck}	25	MPa		f_{yk}	500	MPa
	f_{ctm}	2.2	MPa		E_s	210	GPa
	E_{cm}	31	GPa				
Concrete	Design values			Steel	Design values		
	f_{cd}	14	MPa		f_{yd}	435	MPa
	f_{ctm}	2.2	MPa		E_{sd}	183	GPa

Decided M-N capacity for the column

Calculation in stages A to D

Stage A

$$\lambda f_{cd} b x_A^2 + A_{s2}\varepsilon_{cu}E_s\left(x_A - d_2\right) - A_{s1}f_{yd}x_A = 0 \Leftrightarrow$$

$$0.8\cdot 14\cdot x_A^2 + 1256\cdot 0.0035\cdot 210\cdot 10^3\left(x_A - 40\right) - 1256\cdot 435\cdot x_A = 0$$

$$\Rightarrow x_A = 60.76\,mm$$

$$\varepsilon_{s1,A} = \left(\frac{d}{x_A} - 1\right)\varepsilon_{cu} = \left(\frac{360}{60.76} - 1\right)\cdot 0.0035 = 0.017$$

$$\varepsilon_{s2,A} = \frac{x_A - d_2}{x_A}\varepsilon_{cu} = \frac{60.76 - 40}{60.76}\cdot 0.0035 = 0.00119$$

$$\sigma_{s2,A} = \min\left\{\varepsilon_{s2,A}E_s; f_{yd}\right\} = \min\left\{0.00119\cdot 210000; 435\right\} = 218\,MPa$$

$$M_{u,A} = \lambda x_A f_{cd} b\left(d - d_{pg} - \frac{\lambda}{2}x_A\right) + A_{s2}\sigma_{s2,A}\left(d - d_2 - d_{pg}\right) + A_{s1}f_{yd}d_{pg} =$$

$$= 0.8\cdot 60.76\cdot 14\cdot 400\left(360 - 160 - 0.8\cdot\frac{60.76}{2}\right) +$$

$$+1256\cdot 218\cdot\left(360 - 40 - 160\right) + 1256\cdot 435\cdot 160 = 179.2\,kNm$$

Stage B

$$x_B = \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{sy}}d = \frac{0.0035}{0.0035 + 0.00238}360 = 214\,mm$$

$$\varepsilon_{s2,B} = \frac{x_B - d_2}{x_B}\varepsilon_{cu} = \frac{214 - 40}{214}\cdot 0.0035 = 0.00285$$

$$\sigma_{s2,B} = \min\left\{\varepsilon_{s2,B}E_s; f_{yd}\right\} = \min\left\{0.00285\cdot 210000; 434\right\} = 435\,MPa$$

$$N_{u,B} = \lambda x_B b f_{cd} - A_{s1}f_{yd} + A_{s2}\sigma_{s2,B} =$$

$$= 0.8\cdot 214\cdot 400\cdot 14 - 1256\cdot 435 + 1256\cdot 435 = 959.65\,kN$$

$$M_{u,B} = \lambda x_B b f_{cd}\left(d - d_{pg} - \frac{\lambda}{2}x_B\right) + A_{s1} f_{yd} d_{pg} + A_{s2}\sigma_{s2,B}\left(d - d_1 - d_{pg}\right)$$

$$= 0.8 \cdot 214 \cdot 400 \cdot 14\left(360 - 160 - 0.8 \cdot \frac{214}{2}\right) -$$

$$-1256 \cdot 435 \cdot 160 + 1256 \cdot 435 \cdot (360 - 40 - 160) = 284.62\, kNm$$

Stage C

$$\varepsilon_{s2,C} = \frac{d - d_2}{d}\varepsilon_{cu} = \frac{360 - 40}{360} \cdot 0.0035 = 0.00311$$

$$\sigma_{s2,C} = \min\left\{\varepsilon_{s2,C} E_s ; f_{yd}\right\} = \min\{0.00311 \cdot 210000; 434\} = 435\, MPa$$

$$N_{u,C} = \lambda d f_{cd} b + A_{s2}\sigma_{s2,C} = 0.8 \cdot 360 \cdot 400 \cdot 14 + 1256 \cdot 435 = 2159\, kNm$$

$$M_{u,C} = \lambda d b f_{cd}\left(d - d_{pg} - \lambda\frac{d}{2}\right) + A_{s2}\sigma_{s2,C}\left(d - d_2 - d_{pg}\right)$$

$$= 0.8 \cdot 360 \cdot 400 \cdot 14 \cdot \left(360 - 160 - 0.8 \cdot \frac{360}{2}\right) +$$

$$+1256 \cdot 435 \cdot (360 - 40 - 160) = 177.78\, kNm$$

Stage D

$$\varepsilon_{s1,D} = \varepsilon_{s2,D} = \varepsilon_{cök,D} = \varepsilon_{cuk,D} = \varepsilon_{cu}$$

$$\sigma_{s1,D} = \sigma_{s2,D} = f_{yd}$$

$$N_{u,D} = f_{cd} A_c + f_{yd}\left(A_{s1} + A_{s2}\right) = 14 \cdot 400 + 435(1256 + 1256) =$$

$$= 3298\, kN$$

Estimate the confinement effect

Choose FRP sheets with a thickness of 0.17 mm (sheet ca 300g/m^2), material properties can be found in table H3.4.

Table H3.4 Characteristic and design values for FRP.

	Characteristic values			Design values		
FRP	ε_{fk}	19	‰	ε_f	14.1	‰
	E_{fk}	290	GPa	E_f	214.8	GPa

Step 1. Calculate the geometrical parameters

$$D = \sqrt{b^2 + h^2} = \sqrt{400^2 + 400^2} = 566\, mm$$

$$\frac{A_{ce}}{A_c}=\frac{1-\frac{\left[\frac{b}{h}(h-2r_c)^2+\frac{h}{b}(b-2r_c)^2\right]}{3A_g}-\rho_g}{1-\rho_g}=$$

$$=\frac{1-\frac{\left[\frac{400}{360}(400-2\cdot 30)^2+\frac{400}{360}(400-2\cdot 30)^2\right]}{3\cdot 400\cdot 400}-\frac{1256}{400\cdot 400}}{1-\frac{1256}{400\cdot 400}}=0.507$$

$$\kappa_a=\frac{A_{ce}}{A_c}\left(\frac{b}{h}\right)^2=0.507\cdot\left(\frac{400}{400}\right)^2=0.507$$

$$\kappa_b=\frac{A_{ce}}{A_c}\left(\frac{b}{h}\right)^{0.5}=0.507$$

$$\kappa_e=0.55$$

$$\varepsilon_{fe}=\min\left(0.004,\kappa_e\varepsilon_f\right)=0.004$$

Step 2. Calculate the confinement pressure

The process is iterative. Here the number of layers of FRP is first assumed. In this example two iterative steps are needed..

Choose number of layers, n=6.

Calculate radial stress::

$$f_l=\frac{2E_f n t_f \varepsilon_{fe}}{D}=\frac{2\cdot 214815\cdot 6\cdot 0.17\cdot 0.004}{566}=3.1\,MPa$$

Calculate the confinement effect:

$$f_{cd,c}=f_{cd}+\alpha_{f,c}3.3\kappa_a f_l=14+0.95\cdot 3.3\cdot 0.507\cdot 3.1=18.92\,MPa$$

With this new value of the compressive strength for the confined cross section a new calculation for the M-N interaction curve must be carried out – same process as earlier but now with, $f_{cd,c}$, instead of f_{cd}.

The strengthening effect by confine the column with 6 layers carbon fibre sheets (300 g/m^2) is shown in figure C.2. It can be noted that 6 layers is not enough for a 15% increase of the load. Therefor we have to make a new calculation, this time with *n* = 8. This then gives the new radial pressure and a new compressive strength according to:

$$f_l=\frac{2E_f n t_f \varepsilon_{fe}}{D}=\frac{2\cdot 214815\cdot 8\cdot 0.17\cdot 0.004}{566}=4.13\,MPa$$

$$f_{cd,c}=f_{cd}+\alpha_{f,c}3.3\kappa_a f_l=14+0.95\cdot 3.3\cdot 0.507\cdot 4.13=20.56\,MPa$$

Then we have to repeat the steps again with the newly calculated compressive strength of the concrete, $f_{cd,c}$. In this case the column can take the 15% increase, see figure H.2.3.

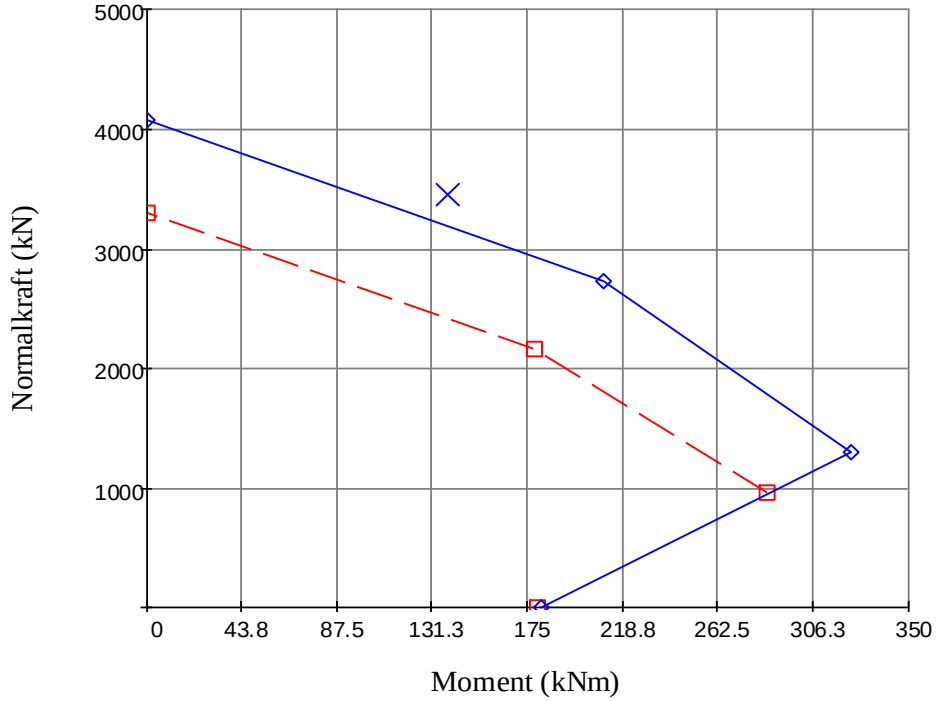


Figure H3.2. M-N interaction for the non (red) and the strengthen column (blue). The cross indicate the needed capacity.

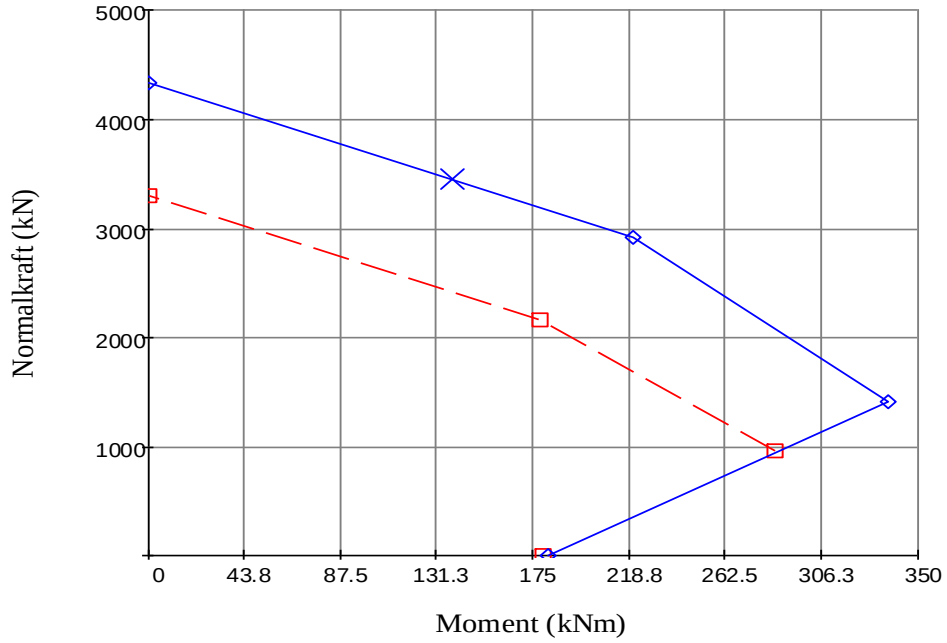


Figure H3.3. M-N interaction for the non (red) and the strengthen column (blue). The cross indicate the needed capacity

Step 3. Check the strain level in the concrete

The strain level in the concrete must be investigated to that it do not exceed 1%. This to avoid exaggerated crack development, which might lead to reduced structural integrity. The actual strain level is calculated as:

$$\varepsilon_{cu,c} = \varepsilon_{c2}\left(1.50 + 12\kappa_b \frac{f_l}{f_{ck}}\left(\frac{\varepsilon_{fe}}{\varepsilon_{c2}}\right)^{0.45}\right) =$$

$$= 0.002\left(1.5 + 12 \cdot 0.507 \cdot \max\left(0.08, \frac{4.13}{14}\right)\left(\frac{0.004}{0.002}\right)^{0.45}\right) = 0.008 \leq 0.01$$

The strain level in the concrete is then within the approved level after strengthening with 8 layers of carbon fibre sheets.