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ARCHES

Assessment and Rehabilitation
of Central European Highway Structures

**SPECIFIC TARGETED RESEARCH PROJECT
SUSTAINABLE SURFACE TRANSPORT**

ARCHES-MG-DE 15 **FINAL ACTIVITY REPORT**

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SIXTH FRAMEWORK PROGRAMME
Sustainable Surface Transport

Assessment and Rehabilitation
of Central European Highway Structures

DELIVERABLE D 15

FINAL ACTIVITY REPORT

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1 EXECUTIVE SUMMARY

1.1 Project objectives

The strategic objective of the Project is to reduce the gap in the standard of highway structures between the Central and Eastern European Countries (CEEC), including New Members States (NMS), and the rest of the EU, in a sustainable way. This will be achieved by developing appropriate tools and procedures for a more efficient assessment, and faster, cost-effective, and long lasting rehabilitations (repair or strengthening) of sub-standard highway structures.

To achieve its scientific and technological objectives, this project focuses on structural assessment and monitoring, strategies to prevent deterioration and optimum rehabilitation of highway structures by complementary techniques. It is organised in 4 technical work packages, with the following conceptual approach.

- Optimise the use of existing infrastructure through better safety assessment and monitoring procedures which will avoid interventions, i.e., avoid unnecessarily replacing or rehabilitating structures that are in fact perfectly safe (WP 2).
- Monitor and prevent corrosion of existing reinforcement and develop innovative new reinforcement materials that are highly resistant to corrosion (WP 3).
- Strengthen the infrastructure of bridges by means of bonded reinforcements (WP 4)
- Harden highway structures with Ultra High Performance Fiber Reinforced Concretes applied in severely exposed zones to dramatically increase their durability (WP 5)

1.2 Contractors involved

The whole Consortium consists of 12 Partners involved in full spectrum of Project activities. One of the Partners Forum of European National Highway Research Laboratories (FEHRL) affiliates a several institutes, FEHRL's members, which play a minor role in the Project. The full list of Partners is given below.

- | | |
|---|-----------------|
| ▶ IBDiM – Road and Bridge Research Institute | POLAND |
| ▶ ZAG – Slovenian National Building and Civil Engineering Institute | SLOVENIA |
| ▶ CDV – Transport Research Center | CZECH REPUBLIC |
| ▶ UPC – Technical University of Catalonia | SPAIN |
| ▶ EPFL – Ecole Polytechnique Fédérale de Lausanne | SWITZERLAND |
| ▶ UCD – University College Dublin | IRELAND |
| ▶ FEHRL – Forum of European National Highway Research Laboratories; | BELGIUM |
| ○ including | |
| • TECER | Estonia, |
| • Central Roads and Bridges laboratory (CRBL) | Bulgaria |
| • Civil Engineering Institute of Croatia (IGH) | Croatia |
| • Arsenal Research | Austria |
| • Laboratories Central des Ponts et Chaussées | France |
| ▶ Leggedoor Concrete Repair | THE NETHERLANDS |
| ▶ Autostrade per l'Italia | ITALY |
| ▶ University of Zagreb | CROATIA |
| ▶ Salonit Anhovo | SLOVENIA |
| ▶ TNO | THE NETHERLANDS |

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1.4 Work performed during the Project execution (1.09.2006-31.08.2009)

As mentioned previously the research work was organized within the four Workpackages devoted to different objectives. The additional Workpackage was devoted to the dissemination of results. Below the spectrum of activities of particular Workpackages and their tasks is presented.

1.4.1 Structural assessment and monitoring

As the Work Package is quite multiple and covers a broad area of activity the performed activity will be presented within the appropriate subtasks.

Bridge traffic load monitoring

- Development of methods and techniques to assess the real traffic loads on bridges,
- Development of an advanced Bridge Weigh-In-Motion algorithm based on Tikhonov Regularisation that will improve the collection and monitoring of traffic data.
- Analysis an updating database of truck weights from Slovakia,
- A simplified model for the calculation of site-specific characteristic load traffic action previously developed in SAMARIS project has been checked with the new traffic data obtained from the NMS.
- The development of an advanced Bridge Weigh-In-Motion algorithm based on Tikhonov Regularisation that will improve the collection and monitoring of traffic data has been finished
- Traffic data has been recorded and analyzed in 5 European countries, one from the West (The Netherlands) and 4 from CEEC (Czech Republic, Poland, Slovakia and Slovenia). The final result has been the calculation, via simulation, of the characteristic load effects and the reduction factors by bridge class to be considered in the assessment of existing bridges in CEEC,
- A simplified traffic load modelling procedure was proposed by ARCHES,
- The Tikhonov regularisation as been developed and implemented in the analysis of WIM data. The new method has been successfully applied in the project.

Bridge performance monitoring

- State of the art of structural health monitoring technologies was examined,
- Review of the monitoring systems, which consist of several components that ensure the acquisition of structural response, signal processing and communication of evaluated results to the users was prepared,

- Analysis of the usage of damage prognosis in the estimation of the remaining useful of a structural system were conducted.
- Derivation of relevant results not only for the diagnostic (actual state of the bridge) but also for the prognosis (future state),
- Review reviewed damage indicators that can be used for bridge diagnosis was prepared,
- The list of ongoing structural health monitoring projects in USA and Asia. has been completed.
- A state-of-the-art on monitoring techniques and Structural Health Monitoring experiences in Europe and the rest of the world has been prepared and included in D08,
- The procedure implemented in Slovenia for the correlation between the results of bridge inspection (Damage Index) and the Capacity Reduction factor has been also checked in the case of other countries as Poland,
- Preparation of the Deliverable D08 Recommendations on the use of results of monitoring on bridge safety assessment and maintenance

Acoustic Emission

- Laboratory tests on plain and reinforced concrete beams loaded in bending were prepared and executed,
- The pull out of R-bars tests and compressive tests on concrete cubes were performed,
- Preparation for the first on-site application.
- The laboratory tests on the reinforced concrete girders removed from the Gameljščica bridge and sent to the ZAG laboratory were prepared and executed,
- The investigation on the use of AE on plain and reinforced concrete specimens loading in bending continued,
- The proof load test was performed on the bridge over the Gameljščica river in Gameljne, foreseen to be demolished,
- Laboratory as well as “in situ” tests using acoustic emission were carried out both by ZAG and IBDIM,
- AE has been used in the execution of the proof-load testing of Barcza bridge in Poland,

Soft load testing

- The implementation of soft load task has been carried out a load tests in a bridge close to Ljubljana.
- ZAG has carried out soft load testing in 6 bridges in The Netherlands. Some additional tests have been also performed in Slovenia.
- The previous results achieved in project SAMARIS on soft load testing have been fully validated in ARCHES by the application of the test to more than 20 bridges in Slovenia.

Diagnostic load testing

- Elaboration of information form to collect data from existing load testing results and analytical calculations of any type and size bridges,
- Elaboration of the Internet application for collecting load test data,
- Preparation the data from load testing results and analytical calculations of bridges:
 - First part - Bridges in Warsaw (about 40 examples)
 - Input the data to the Internet application (10 bridges - 2007-09-25)
- Preparation the first draft of text description of D 07: Internet database of load test results and analytical calculations,
- Preparation the input to the first draft of the Recommendation on the use of Diagnostic Load Testing

- Elaboration of a new version of the Internet application for collecting load test data,
- Preparation the data from 60 examples of load testing results and analytical calculations of bridges: (100 examples – the first and the second year together),
- Inputting the data to the Internet application (about 50 examples),
- Preparation the common data base with all partners permitted reading access.
- The internet database of load testing results and analytical calculations has been developed,
- Preparation of the Deliverable D07 Internet database of load test results and analytical calculations.

Proof load testing

- The state-of-the-art on proof load tests in bridges carried out worldwide,
- The state-of-the-art on worldwide existing recommendations and guidelines for proof load tests has found interesting documents in Germany (DAfStb) and USA (AAHSTO),
- A literature search on the use of acoustic emission in tests up to failure in bridges and other structures has started,
- The development of a method to define the target proof load has started with the analysis of the candidate bridge for testing in Poland.
- The development of a method to define the target proof load,
- A preliminary study has been applied to two different traffics from The Netherlands and Slovakia.
- A complete set of proof load factors for 5 European countries (The Netherlands, Czech Republic, Poland, Slovakia and Slovenia) have been calculated based on the actual traffic conditions in these countries as obtained by WIM techniques developed in task 2.1.1,
- A proof load test in a real structure has been carried out (Barcza bridge, Poland),
- Preparation of the Deliverable D16 Recommendations on the use of soft, diagnostic or proof load testing .

Reducing dynamic loading of bridges

- Comprehensive literature review has been gathered,
- The influence of the road profile has been investigated using a quarter-car model travelling over a bridge,
- Assessment Dynamic Ratio has been defined here as the ratio of characteristic total load effect to characteristic static load effect and it is proposed within ARCHES as a method to characterise the dynamics of a bridge for a given return period.
- The values of the Dynamic Amplification Factor (DAF) and Allowance Dynamic Ratio (ADR), defined as the ratio of characteristic total load effect to characteristic static load effect have been obtained considering the most important variables involved in the process,
- Preparation of the Deliverable D10 Recommendations on dynamic amplification allowance in assessment of bridges.

Systematic decision making processes associated with maintenance and reconstruction of bridges

- The list preparation of potential recipients of the BMS questionnaire (NMS and CEEC's),
- Preparation repairing the first draft of the BMS questionnaire (prepared for distribution among NMS and CEEC's),

- Designing the structure of the national report (more detailed information from WT partners and representatives of other countries),
- The questionnaire prepared during the previous period was spread among the contacts of WT members in new member states.
- The recommendations to elaborate a common Bridge Management System (BMS) for the NMS and CEEC have been proposed,
- The answers to the questionnaire on decision making process associated with maintenance and reconstruction of bridges from 14 countries were received and analyzed (Bulgaria, Croatia, Czech Republic, Estonia, France, Germany, Hungary, Italy, Latvia, Serbia, Slovakia, Slovenia, UK, Ukraine),
- Preparation of the Deliverable D 09 Recommendations on systematic decision making processes associated with maintenance and reconstruction of bridges.

1.4.2 Prevention of corrosion

This work package is multi disciplinary as well and covers the issues related to the corrosion of reinforcing steel and concrete. The activity divided into three task covered the following items.

Validation and application of low-alloy steel

- After a thorough research of available low-alloyed steel for reinforcement application of the market following steel types and steel producers have been chosen,
- Exposure site at the Adriatic coast (near city of Rijeka, Croatia) was selected
- Laboratory testing in simulated pore solution generally consisted of two main groups of electrochemical measurements were conducted:
 - electrochemical impedance spectroscopy (EIS)
 - potentiodynamic polarisation scans.
- Milestone 10 “Report on laboratory results (low-alloy steel, corrosion probes)” was prepared according to the schedule.
- The legal permissions have been issued for testing field at Adriatic coast beside the Krk bridge and preparation of the has started.
- The tests result were compared with the results of the “steel in concrete” tests and analysed thoroughly for the final reports and the D11 Recommendations for the use of corrosion resistant reinforcement
- At UZ corrosion behaviour of different steel types embedded into smaller concrete specimens (“lollipops”) were performed in water solutions with different chloride content and pH values using potentiostatic anodic polarization and electrochemical impedance spectroscopy, All tests were finished as planed.
- Concrete specimens with embedded low-alloyed steels and ER probes, developed and manufactured within the WT 3.3, were exposed to carbonation and wetting from the top by chlorides. Electrochemical potential measurements and the galvanostatic pulse technique were performed
- 6 types of reinforcing steel were embedded in the columns, while for each type of reinforcing steel 3 columns were produced, which means that in total 18 columns were cast. The test site will be monitored in the future and the results reported elsewhere later.
- preparation of the Deliverable D11 Recommendations for the use of low-alloy steel.

Development and application of cathodic protection system

- Preparation of concrete specimens for laboratory testing of the cathodic protection (CP) system (especially the anodic coating) exposed to simulated environmental conditions is in the

- execution phase,
- The on site application of CP two structures were chosen
- The CP system was successfully applied in Slovenia,
- The first draft of the recommendation for the use of CP systems (list of content) was prepared and discussed among the WP partners.
- The project intensive monitoring of the CP site was performed. Based on the measurement results the current was optimised and the theoretical model used for the CP system simulation was evaluated
- The CP system was successfully applied in Poland,
- Preparation of the Deliverable D12 Recommendations for the use of Cathodic Protection systems.

Development/modification of corrosion monitoring system

- to obtain wider experience a number of concrete specimens with embedded electrodes/probes were built,
- the procedure for the installation of the probes into the testing field has already been defined.
- the ER probes were imbedded in concrete samples and tests sites of WT 3.1 and WT 3.2 and were used for corrosion monitoring.

1.4.3 Strengthening with FRP glued strips

This task devoted to the structure elements strengthening has reported following activity:

- The theoretical basis for strengthening structures with the use of prestressed FRP elements were prepared.
- Application of prestressed FRP materials on concrete beam girders,
- Stress-strain models of FRP-confined concrete columns were prepared,
- Application of prestressed FRP materials on real concrete beams of the Seroczyn bridge in Poland,
- Finishing of the laboratory investigation on confined columns,
- Preparation of the Deliverable D13 Recommendations for prestressed externally glued FRP strips

1.4.4 Harden Structures to last with UHPFRC

Repairing and hardening the structures were the essential goals of this work package.

- The UHPFRC based on a local components investigation first phase was accomplished in Slovenia
- Technology transfer of UHPFRC production and testing to Polish partner of the work package,
- Development a UHPFRC with components available in Poland, validate this material,
- Optimisation processing technology of UHPFRC: tolerance to slopes of up to 5 %, surface rendering, validation of jointing techniques, effect of climatic conditions at casting,
- Determination of fibre orientation and distribution in UHPFRC with different methods, in view of non- destructive analysis,
- Determination the effect of surface roughness of substrate on structural performance of UHPFRC: numerical simulations and experimental tests on composite specimens,
- Choice of on site applications for pilot tests was done. (Milestone M08).
- Performing of a full scale trial test of Slovene based UHPFRC mixes and validate slope

tolerance up to 5 %.

- Optimize processing technology of UHPFRC: for surface rendering, validation of jointing techniques, effect of climatic conditions at casting.
- Performing of full scale application on a bridge in Slovenia with the Slovene UHPFRC mixes developed in years 1 and 2.
- Realize a film and a DVD on this application.
- Drafting of deliverable D06: Recommendations for the tailoring of UHPFRC recipes for rehabilitation (All) and D14: Recommendations for the use of UHPFRC in composite structural members.

1.4.5 Dissemination

During the Project activity several dissemination activities were performed:

- Polish National workshop in Poland was organised in Kielce in May 2007,
- Hungarian National workshop took place on 11 September 2008 in Keszthely-Heviz in Hungary,
- Czech National Workshop was held in Brno on 25 November 2008,
- Slovenian National Workshop took place in Bled on 6 - 7 May 2009,
- Ukrainian National Workshop was held in Kapitanivka Villiage near Kiev on 21 May 2009,
- Estonian National Workshop took place in Tallinn on 5 June 2009,
- ARCHES project was disseminated in the event as part of FEHRL's EC projects during the European Cities of Science, which was an event under the French Presidency of the European Union,
- ARCHES project was disseminated in the event as part of FEHRL's EC projects during the ERTRAC Conference, 26 January 2009, Brussels
- ARCHES project was disseminated in the event as part of FEHRL's EC projects during the 88th Transportation Research Board meeting in Washington, 11 – 15 January 2009,
- ARCHES project was disseminated in the event as part of FEHRL's EC projects at the Research Connection 2009 (conference and exhibition) was organised in conjunction with the Czech Presidency of the European Union, 7 – 8 May, 2009, Prague,
- A presentation of the ARCHES project was given by Emmanuel Denarie at the SIMBA II Russia Workshop on 27 – 28 November in Moscow,
- The ARCHES and SPENS Final Seminar took place in Ljubljana, Slovenia on 27 – 28 August 2009.
- A special Technical Excursion to some of the ARCHES and SPENS test sites took place on 28 and 29 August.

2 SECTION I PROJECT OBJECTIVES AND MAJOR ACHIEVEMENTS DURING THE REPORTING PERIOD

2.1 Overview of general project objectives

The strategic objective of the Project is to reduce the gap in the standard of highway structures between the Central and Eastern European Countries (CEEC), including New Members States (NMS), and the rest of the EU, in a sustainable way. This will be achieved by developing appropriate tools and procedures for a more efficient assessment, and faster, cost-effective, and long lasting rehabilitations (repair or strengthening) of sub-standard highway structures.

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assessment and monitoring, strategies to prevent deterioration and optimum rehabilitation of highway structures by complementary techniques. It is organised in 4 technical work packages, with the following conceptual approach.

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- Monitor and prevent corrosion of existing reinforcement and develop innovative new reinforcement materials that are highly resistant to corrosion (WP 3),
- Strengthen the infrastructure of bridges by means of bonded reinforcements (WP 4),
- Harden highway structures with Ultra High Performance Fiber Reinforced Concretes applied in severely exposed zones to dramatically increase their durability (WP 5).

In practice the Project outcome are supposed to create a set of recommendations for the wide use of knowledge achieved through the investigation activity. Several deliverables are designed in a form of recommendations – these are recommendations for use new materials and technologies in monitoring, corrosion protection strengthening and hardening of the structures.

2.1.1 Deliverables

Table 1 presents the deliverables of the Arches Project. All the Deliverables have been accomplished and are presented as the official project documents on the ARCHES dedicated web page <http://arches.fehrl.org/>

Table 1 List of ARCHES deliverables

Delive rable	Deliverable title
01	Project Internet site
02	Brochure presenting the project
04	Report on Final Seminar
05	Yearly progress reports
06	Recommendations for the tailoring of UHPFRC recipes
07	Internet database of load test results and analytical calculations
08	Recommendations on the use of results of monitoring on bridge safety assessment and maintenance
09	Recommendations on systematic decision making processes associated with maintenance and reconstruction of bridges
10	Recommendations on dynamic amplification allowance in assessment of bridges
11	Recommendations for the use of low-alloy steel
12	Recommendations for the use of Cathodic Protection systems
13	Recommendations for prestressed externally glued FRP strips
14	Recommendations for the use of UHPFRC for composite structural members
15	Executive summary report of the Project
16	Recommendations on the use of soft, diagnostic or proof load testing

2.1.2 Milestones

Table 2 summarises the milestones of the ARCHES project which were passes during the three year activity. .

Table 2 List of ARCHES milestones due in year 3

Milestone	Milestone title
01	QA auditors appointed
02	Selection of on site applications (CP systems)
03	Internet application for collecting load test data available
04	Selection of on site application (low-alloy steel)
05	Choice of on site applications for pilot tests. Series 1 (country 1)
06	Development of UHPFRC from local components
07	Stress-strain models of FRP-confined concrete columns
08	Choice of on site applications for pilot tests. Series 2 (country 2)
09	Mid-term follow-up. Specified results delivered. Financial and man-month expenditures in compliance with those planned
10	Report on laboratory results (low-alloy steel, corrosion probes)
11	All load tests completed
12	Report on pilot tests of application
13	Final symposium. Preparations for the final symposium completed.

3 SECTION 2 WORK PACKAGE RESULTS

3.1 WP2 – Structural assessment and monitoring

3.1.1 Starting point of work

Start date of WP2 was Month 1 of the project.

3.1.2 Objectives

The main objective of WP2 is to provide recommendations and guidance for implementation of optimized bridge assessment tools in NMS and CEEC. The recommendation will deal with monitoring, load testing of different types from soft load testing up to the proof one (experiment in Poland), dynamic impact on bridges and Bridge Management System development.

3.1.3 Bridge performance monitoring - Recommendations on the use of results of monitoring on bridge safety assessment and maintenance

Introduction

The main objective of the structural assessment and monitoring part of the ARCHES project is to develop techniques for optimal bridge assessment that are appropriate for the use in Central & Eastern European Countries. Due to increased traffic volumes and structural

deterioration, many of the existing bridges seem not to satisfy the requirements for safe operation. However, replacement of such large part of the bridge stock would be very expensive. The goal of this project is to provide help in assessment of existing bridges using modern monitoring technologies, which should give more accurate estimate of traffic loads on one hand and structural performance on the other hand.

According to the findings of previous European and other international projects, there are great variations in the composition of actual heavy traffic from country to country. The real traffic loading conditions of highway structures in Central & Eastern European Countries are mostly unknown. If it can be proven that the traffic loading on a bridge is less than was previously thought, it is possible to greatly extend the safe working lives of existing bridges and to extend the time interval between interventions. The results of traffic load assessment are presented in chapter 0 of the Deliverable D 08.

The load carrying capacity of many highway structures is not known either, especially for very old bridges where the design and construction documents are not available. Load testing of bridges has considerable potential to improve knowledge of load carrying capacity but is currently practiced primarily for new bridges. Alternatively, monitoring techniques can be used to provide information useful for bridge assessment. Monitoring can be considered as an additional assessment tool, an extension of visual inspection. The results of monitoring use in structural assessment are presented in chapter 0 of the Deliverable D 08.

Traffic load Assessment

Traffic loads in Central and Eastern European Countries

The Eurocode load model for highway traffic effects is based on the load effects with a 5% probability of exceedance in a design life of 50 years, which is effectively the same as the load effect with a return period of 1000 years. A few heavily trafficked sites in Europe have a high frequency (a few per day) of extremely heavy (> 100 tonne) vehicles. In such situations, it was assumed in the past that the traffic loading could be approaching the levels specified in the Eurocode. It has been found in ARCHES that load levels are much higher than was assumed. In a measurement site in Netherlands, characteristic load effects were found to be 20% to 50% in excess of the levels suggested by the Eurocode. However, vast majority of Europe's highway bridges are subject to considerably less traffic loading than this site.

To investigate real traffic loads in Central and Eastern European (CEE) countries, Weigh-in-Motion (WIM) measurements were carried out on motorway bridges in Slovakia, Poland, Slovenia and the Czech Republic. The results were compared to data from the Netherlands as a reference. The measured traffic, particularly in the Netherlands, includes many very heavy vehicles and gives an insight into what the future may hold for other less densely trafficked locations. The heaviest vehicle that was captured by measurement was in Netherlands with a gross weight of 166 tons. In Slovakia, the heaviest vehicle had 117 tons.

The load effects from the measured traffic were investigated for bridges with spans between 15 and 45 m. For these bridges the free-flowing type of traffic is more critical than the congested traffic. The calculated characteristic load effects were compared to traffic load models LM1 and LM3 defined by the Eurocode. Results were calculated for two extreme values of lane factor. The lane factor defines, how traffic in different lanes contributes to stress a structural element. If each lane will contribute equally to its stress, then the 'lane factor' is defined as unity. On the other hand, if the element is at the edge of one lane, vehicles in the other lane may have little effect and the lane factor will be significantly less than unity. The characteristic load effects revealed to be highly sensitive to the value of lane factor.

Recommendations for design of new bridges

Eurocode Load Model LM1

The four Central European countries showed fairly similar results, and are lower than the Netherlands. There are significant excesses – up to about 20% – over the Eurocode load model LM1, particularly in the case of shear in bridges with low lane factors. Motorway data was used in these calculations for bridges subject to bi-directional traffic. This approach is applicable because the numbers of trucks per day was at a level that could be experienced on a non-motorway without resulting in congestion. It can be concluded that, while there is no evidence to suggest that bi-directional traffic has already reached or is exceeding Eurocode levels, it has the potential to do so, if truck traffic on such roads reaches levels currently being recorded in adjacent motorways.

It can be concluded that the Eurocode Normal load model for the design of bridges is less conservative than previously thought and may become less so if the frequencies of extremely heavy vehicles increase. There is a need for greater control, perhaps by Global Positioning Systems tracking, of these extreme vehicles. In the absence of such control, the Eurocode LM1 should be revised to specify greater loading. Furthermore, the relative loadings in each lane specified in the Eurocode should be revised.

There should be less concern for the existing bridge stock as the actual probabilities of exceedance on non-motorway bridges are still likely to be well below the acceptable range.

Eurocode Load Model LM3

Eurocode Load Model 3 (“LM3”) provides for a set of standardised vehicle models (permit vehicles), assumed here to be traveling at normal speeds, which is the case for the measured traffic. Also for this load model, the calculated characteristic load values showed exceedance of loads as defined by Eurocode. For each country, a minimum required load was calculated that would ensure a conservative bridge design under current traffic conditions (see appendix A of the Deliverable D 08). These values are given in Table 3 and are recommended for implementation in National Application Documents. However, if conservatism is to continue, the frequency of special permit vehicles would need to be controlled and prevented from reaching the levels recorded in the Netherlands. Unless steps are taken to carefully control the numbers of permits issued during the bridge lifetime, National Application Documents for the design of new bridges should specify the 1800/200 vehicle (in both lanes) in all CEE countries. This level of loading is considerably greater than that being specified in most countries at the present time.

Table 3 Minimum LM3 model required for each site for all load effects

Site	Lane Factors	LM3 Required
Netherlands	High	1200/200
	Low	1800/200
Czech Republic	High	1200
	Low	1500
Slovenia	High	900
	Low	1500

Poland	High	1200
	Low	1200/200
Slovakia	High	900
	Low	1200/200

Recommendations for assessment of existing bridges

Conservatism in the design of new bridges can be justified by the relatively low cost implications and the fact that it allows for possible future increases in traffic load. However, for existing bridges, the cost of conservatism is much greater as it may result in the premature replacement or rehabilitation of structures. As a result, lesser levels of safety can be justified for the assessment of existing structures. While new bridges in Europe are designed for a return period of 1000 years, the ARCHES project recommends a return period for assessment of just 50 years which is equivalent to a 10 % chance of exceedance in 5 years.

In this study, the WIM data was collected from motorway sites. The traffic volumes are shown in Table 4.

Table 4 Truck traffic volumes used in simulation

Country	Site	Site ADTT ^a
Netherlands	Woerden	7 100
Czech Republic	Sedlice	4 750
Slovenia	Vransko	3 300
Poland	Wroclaw	4 000
Slovakia	Branisko	1 100

Note: ^a Annual daily truck traffic in one direction (truck traffic is assumed to be the same in both directions)

The key elements for bridge loading are the weights and frequencies of the extreme vehicles. If it is assumed that the percentage of these vehicles in the total traffic is the same or less on minor roads than on motorways, then bridges on such roads can be assessed using motorway traffic with a simple adjustment based on the reduced total traffic volume. On this basis a bridge with a total truck volume (based on all categories of trucks) that is, for example, 30% of typical motorway truck volumes, would be assessed for a return period of 30% of the recommended 50 years, i.e., 15 years. Reduction factors are given in

Table 5. The factors for design are based on reduced traffic volumes, whereas the factors for assessment combine reductions in volume with the shorter return period. The reduction factors are calculated for all sites and load effects as there is relatively little variation between sites.

Table 5 Reduction factors for reduced truck volumes and for assessment

Truck volume as % of site ADTT	Design (1000 year)	Assessment (50 year)
10%	0.93	0.83
20%	0.95	0.85

30%	0.97	0.87
40%	0.97	0.88
50%	0.98	0.89
60%	0.99	0.89
70%	0.99	0.90
80%	0.99	0.90
90%	1.00	0.91
100%	1.00	0.91

The Eurocode specifies that each country may apply α -factors to the standard LM1 to reflect local conditions. Table 6 gives average α -factors for all spans considered based on a 1000-year return period, with traffic volumes as measured at each site. These factors can be combined with the appropriate factor from Table 5 to give a site-specific α -factor. As one example, Table 7 gives α -factors for the assessment of a bridge with 50% lower traffic volumes than the measured sites. Further information is shown in appendix A of the Deliverable D 08.

Table 6 Alpha factors for design at full traffic volumes

Lane Factors	Site	Mid-span moment	Shear at supports	Hogging moment
High	Netherlands	0.99	1.07	1.08
	Czech Republic	0.74	0.91	0.84
	Slovenia	0.73	0.87	0.80
	Poland	0.71	0.85	0.79
	Slovakia	0.69	0.83	0.74
Low	Netherlands	1.05	1.39	1.18
	Czech Republic	0.88	1.19	1.00
	Slovenia	0.84	1.15	0.97
	Poland	0.82	1.10	0.93
	Slovakia	0.80	1.09	0.92

Table 7 Alpha factors for assessment with 50% reduction in traffic

Lane Factors	Site	Mid-span moment	Shear at supports	Hogging moment
High	Netherlands	0.88	0.96	0.96
	Czech Republic	0.66	0.81	0.75
	Slovenia	0.65	0.78	0.71
	Poland	0.63	0.76	0.70
	Slovakia	0.61	0.74	0.66
Low	Netherlands	0.94	1.24	1.05
	Czech Republic	0.79	1.06	0.89
	Slovenia	0.75	1.02	0.86
	Poland	0.73	0.98	0.83
	Slovakia	0.71	0.97	0.82

Simplified traffic load modelling procedure

In the case that specific WIM data is available for the bridge under assessment a simplified methodology based on the convolution technique can be used. This technique accounts for a maximum of four trucks to appear on a two lane bridge and is thus limited to individual spans of up to 40-50 metres. Such bridges comprise well over 90% of all bridges in Europe. The convolution procedure is explained in the main body of the document and appendix A. The method is sufficiently accurate for a number of applications. Its main advantage is that it can apply directly the WIM data and that the calculations are by a long way faster compared to the simulation method. It is however not appropriate for longer bridges and for bridges with more than 2 traffic lanes.

The shape of the influence line has an important influence on the results of the convolution application (see Figure 1). In many cases, these very significant differences are, in large part, due to differences in the support conditions between measured and theoretical influence line, i.e., it appears that bridges assumed to be simply supported are in fact exhibiting some resistance to rotation at the supports. Experiences with soft load testing (see Deliverable D16) indicate that differences (savings) between theoretical and experimental influence lines for bending moment can be especially large on shorter and older single-span bridges, where boundary conditions are not known. Furthermore, measured load distribution factors can precisely define the lane factors.

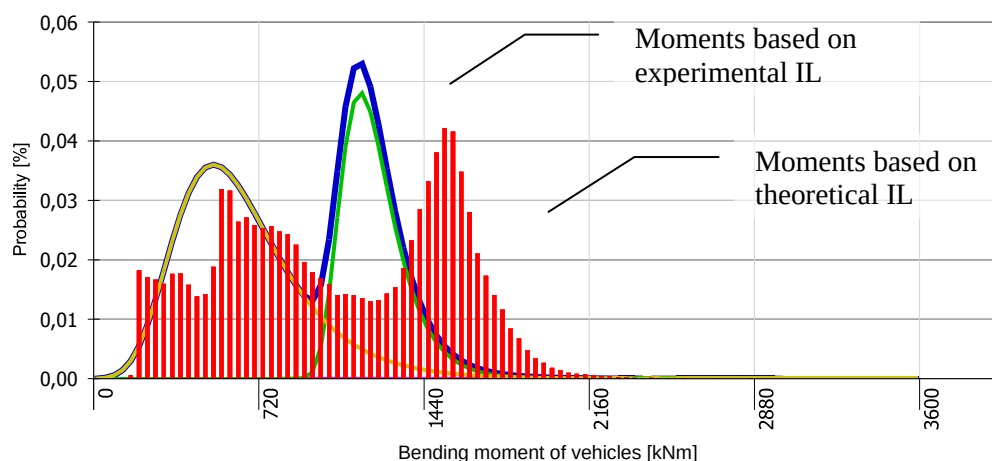


Figure 1-Comparison of bending moments calculated from theoretical and experimental influence lines in driving (above) and overtaking lane (below)

Knowing the real behaviour (influence lines and lane factors) of a bridge has important consequences for optimised bridge assessment. While using theoretical simply supported influence lines and lane factors provides important reserves for design of new bridges that can be used if in the future their condition deteriorates or the traffic conditions change, knowing the experimental influence lines considerably optimises safety assessment of existing bridges and thus prevents from prescribing unnecessary remedial measures on the bridge.

Methodology of Bridge Traffic Load Monitoring

The task of bridge traffic load monitoring includes on one hand capturing the real traffic loads and on the other hand calculation of characteristic traffic load effects, which represent a conservative estimation of the recorded traffic for a particular bridge. The task of capturing traffic loads is accomplished using Bridge Weigh-in-Motion (B-WIM) measurement system.

Characteristic traffic load effects are then estimated by Traffic Load Modelling using data from B-WIM system.

Traffic load measurements

Bridge Weigh-in-Motion uses an instrumented bridge to calculate the axle weights of crossing vehicles. It was first developed in 1979 and is nowadays implemented in commercially available systems. The current B-WIM systems struggle in correct evaluation of situations such as closely spaced axles on longer (over 15-m) bridge spans. In mathematical terms, the equations which relate measured strain on the bridge to the unknown axle weights are ill-conditioned. In the ARCHES project, the solution of the ill-conditioning problem was improved using a numerical technique known as Tikhonov Regularization. This involves an adjustment to the equations which makes them less ill-conditioned. Part of the problem is finding an optimal value of the regularization parameter that defines a degree of adjustment of the original equations. Choice of optimal regularization parameter by L-curve method is described in the project deliverable.

The new technique was tested using dynamic simulations at various vehicle velocities and suspension types. The regularized B-WIM method proved good potential to improve accuracy of the B-WIM system, especially in the case of closely spaced axles and can be recommended for the use in future B-WIM systems. The method is optimal for application on short span bridges (less than 20 m). Application on medium span bridges (up to 40 m) is also possible, but accuracy may be reduced.

Moving Force Identification (MFI) is another evaluation technique of data from B-WIM system. MFI seeks to find the dynamic forces applied by axles to a bridge as vehicle crosses, while B-WIM identifies only static forces. This is a much more challenging problem as considerably more information is being sought from the same quantity of data. The mathematical apparatus is much more complex, using inverse dynamics.

The static axle loads can also be estimated using MFI. While MFI has the potential to provide more accurate static axle loads than regularized B-WIM, the computational effort of MFI is much greater. Further, it requires an accurate finite-element model of the bridge. The MFI method is capable of identifying dynamic axle forces and evaluating dynamic amplification factors.

The inverse dynamics method includes least squares minimisation with Tikhonov regularisation, dynamic programming which provides an efficient solution to the least squares problem and the L-curve method to find the optimal regularisation parameter.

It was tested on data from Vransko Bridge in Slovenia. The axle weights were identified with accuracy ranging from 1 to 7.9%, and dynamic amplification factors were also identified.

Traffic load modelling

The common approach to assessing characteristic bridge traffic loading consists of random simulation of traffic flow and the extraction of the maxima for some period of time (e.g., daily maxima). These observed maxima are then extrapolated to the specified return period, yielding the characteristic value of load effect. A lack of repeatability of results of this current method and uncertainty regarding the accuracy of fitted distributions were recognized as causes for concern. Simulations showed for example results deviations of up to 33% in case of using 25 daily maxima for the extrapolation. It can be concluded that small sample sizes and large extrapolation distances provide highly variable results and should be avoided. The main lack of the extrapolation method is that it can miss very rare load combinations that may appear less than once per year. The situation can be improved by simulation of many years of traffic. The traffic load simulations were performed for 1000 years of traffic.

However, this task is computationally very extensive. It was accomplished through careful program design, with parallel processing using shared memory, and by the use of importance sampling. Multiple processes are run in parallel, with separate processes generating simulated traffic in each lane, and other processes calculating different load effects and gathering block maxima for all event types on bridges of different spans.

Alternatively, the extrapolation method can be used in combination with traffic load simulation covering several years. Longer periods of simulated traffic provide more accurate results. In one particular investigated example, the load effect trend changed after 8 years of simulation, meaning that shorter traffic load simulations would yield incorrect results.

B-WIM measurements are used to derive statistical distributions of truck parameters and arrangement of trucks in the lane. The Gross Vehicle Weight (GVW) and number of axles for each truck are generated using a 'semi-parametric' approach. Up to a certain GVW threshold, where there are enough data to provide a clear frequency trend, the observed (empirical) bivariate distribution for GVW and number of axles is used. Above this threshold, a parametric fit is needed in order to smooth the trend and so that simulations can generate vehicles with weights and axles higher than those observed.

All aspects of the vehicles and the gaps between them have been very carefully modelled in order to achieve a good match between simulations and the very extensive database of measured data. The end result is an excellent match between simulated load effects and those calculated directly from measured traffic. As a result, very good correlation was observed between the simulated data and the measurements.

The described traffic load modelling method was used for calculating characteristic traffic loads presented in chapter 0 and is fully explained in appendix A of the Deliverable D 08.

Structural assessment

The application of monitoring technologies in structural assessment can be recommended according to capabilities of particular technologies. Table 8 summarizes application areas of bridge monitoring technologies. The applications vary from determination of material loading (dead and traffic loads), monitoring of deterioration signs to early warning systems.

Table 8 Summary of possible bridge monitoring applications

Type of bridge	Result	Measurement method	Measurement period
Bridges with external tendons or cables	Dead load effect on tendons or cables	Magnetoelastic method or vibration measurements	Single, periodic or continuous
All bridges	Influence lines of traffic loads	Soft load test	Single measurement
Deteriorating bridges	Monitoring of deterioration increase	Crack meters, strain gauges (vibrating wire or optical fibre), acoustic emission	Continuous or periodic
Bridges with relevant fatigue (e.g. railway bridges)	Fatigue damage accumulation	Strain gauges (foil-type)	Continuous
Bridges of special importance	Early warning system	Vibration, strain, inclination	Continuous

Determination of material loading

Material loading on critical points in the bridge is essential for safety assessment of the bridge. Measurements can be used to determine material loading caused by dead or traffic load.

Loading by dead loads

Possibilities of non-destructive measurements of material loading by dead loads on existing structures are limited.

Available technologies make possible measurements of loading of external tendons or cables. Applicable methods are magnetoelastic method or vibrational measurements. Recommended is the use of magnetoelastic method because it operates under fewer limitations. The cost is relatively low and the method has already been applied in several CEE countries.

Measured dead loads are used to correct the respective values in safety calculations.

Loading by traffic loads

Traffic load effects can be investigated using standard monitoring methods, preferably strain measurements. For improvement in accuracy of traffic load effects, it is recommended to perform a soft loading test. The test measures influence lines at selected points, which are then used instead of theoretical influence lines to calculate effects of traffic load combinations defined in Eurocode. Soft load test uses normal traffic, which implies no exclusion of regular traffic on the structure during the test. Deliverable D16 of this project deals with this topic in detail.

The influence of real traffic that passes the bridge can be measured by strain measurements with a permanent monitoring system. The utilization of such measurements in reliability assessment leads to determination of point-wise reliability indexes at the measured locations. However, the approach is currently still under development and not yet ready for general application. The measurements of real traffic influence can rather be utilized in assessment of fatigue damage accumulation.

Fatigue damage accumulation

Fatigue damage becomes critical in lifetime of bridges of certain types. Especially affected construction types are steel or composite railway bridges. In the case that the lifetime of bridge is endangered by fatigue damage and extension of bridge lifetime is desirable, monitoring techniques can be applied for more accurate assessment of fatigue damage accumulation.

It is recommended to identify the structural details that are most endangered by fatigue damage and to install strain gauges on these locations. Foil-type strain gauges are sufficient for this application and preferred due to their cost, but other strain gauges types are also applicable. The monitoring system should be designed as autonomous permanent system. Traffic effects should be recorded over a period that allows capturing a representative traffic sample. The length of the period may vary by situation; recommended is to measure continuously at least 1 month, preferably several months of traffic.

The fatigue damage accumulation for the measured period is calculated from the data in three steps:

1. Calculation of stresses from measured strains,
2. Counting of stress-cycles using the rainflow counting algorithm,
3. Calculation of fatigue damage accumulation using the Palmgreen-Miner Rule together with respective S-N curves defined in EuroCode.

The remaining fatigue lifetime of particular structural details can be estimated by extrapolation of the measured damage accumulation in time. In case that change in traffic is expected in the future, traffic predictions can also be incorporated into the estimation of remaining lifetime.

Detection of active cracks

Acoustic Emission (AE) is a monitoring method suitable for detection of active cracks using measurements of transient elastic waves on the material surface. It is a promising emerging technology and therefore it is treated here separately from other monitoring technologies.

Application of AE technique is recommended primarily during load tests, but it can also be applied in a continuous long-term monitoring system. The investigations were focused on application on concrete bridges.

The AE detects acoustic waves from various sources: micro and macro crack formation in concrete, concrete crushing, crack surface rubbing, de-bonding of steel rebars or their plastic deformation, and undesired noise.

It is able to recognize relative amount of damage that the structure experiences. The rate of acoustic events is here the primary evaluation parameter.

AE is able to detect near-failure state of bridge, which makes it very useful for application during proof load tests (see deliverable D16). Features that indicate near-failure state are large increase of signal amplitudes and rapid increase of acoustic event rate.

Identification of cracking locations is important in order to detect location of concentrated cracking activities. Locations with concentrated cracking indicate a growing damage region, which is potentially a serious structural damage.

The sensor spacing should be relatively dense due to damping of acoustic waves in concrete. Recommended is to use 200 cm spacing if sensors with 60 kHz resonant frequency are used, and 50 cm spacing if sensors with 150 kHz resonant frequency are used. Non-symmetric sensor placement around the defect improves the crack localization ability. Ideally two or more planar sensor arrays are installed. Parallel to AE equipment, standard monitoring should be also installed to monitor strains or other parameters.

The basic AE parameters obtained by the systems are: rate of acoustic events, acoustic signal duration, signal rising time, number of events, signal amplitude and absolute energy.

If AE is applied during load testing, the information about loads allows obtaining more results from the evaluations. Comparing stress levels to start of AE activity is an indicator of structural quality. The lower the stress at which the AE activity starts, the poorer is the structure. The level of the AE activity during the unloading can be used to evaluate the damage level of the structure. High AE activity during unloading corresponds to high damage levels.

More precise damage assessment can be achieved by Calm and Load ratio evaluation. To evaluate these quantities, complete unloading is needed in the load test. The Calm ratio describes percentage of AE events in unloading phase. Values near zero indicate intact material condition. Load ratio describes load level (relative to max. load) at which AE activity started. The Calm and Load ratio are plotted into a NDIS-diagram, which reveals damage severity in one of 4 categories that correspond with 4 predefined quadrants in the diagram.

Another significant factor is the Felicity ratio, which is the ratio of load at which AE activity starts to previous maximum load. Low Felicity ratio values correspond to poor structural quality. Decreasing Felicity ratio indicated a growing damage.

AE can reveal some information about load history of the bridge. The evaluation is based on Kaiser Effect. Kaiser effect is the absence of detectable acoustic emission until the previous maximum applied load level has been exceeded. The common application of the Kaiser effect

is to determine the maximum prior stress in the structure. In concrete the Kaiser effect is only temporary. After a long period of time the structure can heal itself so that it will produce acoustic emission on subsequent loading at levels lower than previously applied.

For implementation in a structural health monitoring system, the *b*-value analysis appears well suited because of low requirements on computational power and sensor quantity. The *b*-value analysis is based on statistical evaluation of peak amplitudes of AE hits recorded during loading process. The basic concept is that *b*-value (the slope of the frequency versus peak amplitude diagram) drops significantly when stresses are redistributed and damage becomes more localized.

Further information on the AE technique is developed in appendix C of the Deliverable D 08.

Early warning systems

Monitoring systems designed to give early warning are used primarily for detection of structural abnormalities during bridge operation. The monitoring should be continuous. The establishment of early warning systems is recommended for bridges of special importance. Such bridges would produce very high losses in case of failure, functional interruption or late maintenance measures. Therefore an early warning system is recommended as a preventive measure against this case. Due to high costs of continuous monitoring, the implementation is not recommended for majority of bridges.

Extensive structural analysis must precede installation of the early warning system. It is recommended to perform a risk analysis to identify most probable structural damages. The identified damage scenarios should be modelled and sensitivity analysis should be performed. Sensitivity analysis reveals the expected changes in structural response in case of occurrence of particular damage scenarios. Sensitivity analysis is the basis for determining fundamental characteristics of monitoring system: the optimal sensor layout and the ability of damage detection.

The ability to detect damage in particular damage scenarios should be checked with respect to measurement accuracies. The measurements should be normalized in data processing to reduce environmental influences. This improves data accuracy and damage detection ability; in case of a tested bridge in Vienna the improvement was by a factor of 3. Recommended normalization methods are nonlinear regression or autoregressive methods.

Damage identification is carried out by detection of abnormalities in measured structural parameters. Various existing detection methods are applicable. Statistical analysis is a recommended tool. Clustering techniques are equally recommended. A bridge test in France, where different clustering techniques were tested, showed that best results were achieved by hierarchy-divisive clustering method.

The goal of early warning system should be ‘level 1’ detection, i.e. only detection of presence of structural change. Designing monitoring system for ‘level 2’ detection, i.e. localisation of damage, is not recommended due to large costs of both hardware and data processing.

Damage detection ability can be considered as good for damage types that affect global structural parameters, which are eigenfrequencies and strain or inclination along a large part of the structure. Measurements on bridges and damage simulations showed that on steel bridges, loosening of connections can be detected. On prestressed concrete bridges, formation of cracks can be detected. Loss of prestress without crack formation would most likely remain undetected.

Monitoring of deterioration signs

The deterioration of bridges is assessed during regular visual inspections. If deterioration signs have been discovered, the bridge operator has several options of how to deal with it. In

some cases, where repair is not possible or not economical, the most economic solution would be that it would remain in operation as long as possible. If there is high risk that deterioration would reach unacceptable levels until the next visual inspection, installation of appropriate measurement system to monitor the deterioration development on a continual basis is recommended. Alternatively, more frequent visual inspections may also be a solution in some cases.

The deterioration signs that can be monitored include: crack width, strain increase in critical components, stress in external tendons or cables, excessive deformation, vibration, expansion joint gap.

Design of the monitoring system is straightforward, since the location of the structural defects is known (as opposed to early warning systems). Structural analysis must be carried out in order to determine limit values for the monitored quantities. The measurement evaluation is then inexpensive, since it consists of only testing if the monitored parameter does not exceed a given level. Similarly to continuous monitoring in early warning systems, the measurement values should be normalized to compensate environmental influences.

In cases where absolute values of monitored quantities cannot be directly measured (like stress in concrete), their value at beginning of monitoring has to be estimated and the measured relative change is added to give an estimation of the parameter.

Model updating

Model updating is an analysis method based on monitoring results. The basic idea is to update a finite-element model in such a way that it can reproduce measured bridge response. It can be used for purposes of ‘level 3’ damage detection, i.e. identification of damage location and extent.

The use of model updating results can be recommended for identification of suitable locations for detailed inspection. Model updating can provide information about probable damage locations, which should then be inspected on site.

A requirement for application of model updating is a reference measurement from undamaged bridge. Ideally, a measurement that was taken on the investigated bridge in the past would be used. Alternatively, if multiple identical structures are present, measurement differences between the structures can be compared.

Reliability of model updating results is given primarily by number of measured quantities on the bridge, measurement accuracy and choice of updating parameters. In some cases, small variations in the inputs can produce largely different updating results. For this reason, the results should be treated as hints, which are to be verified by inspection.

3.1.4 Diagnostic load testing - Internet database of load test results and analytical calculations

Introduction

Many European countries perform load tests on new and rehabilitated bridges. This extremely useful information is however not used to optimise assessment of existing bridges. The database might help to correlate data on load testing with corresponding results of analytical calculation of different types of bridges from different countries (available data from national resources and from other projects, including tests of bridges before putting them into service, assessment of load carrying capacity of existing bridges and load tests done for research purposes). The database will allow the end users, to judge quickly the behaviour of the

structure under the loading and suggest the structural assessment method to be used – computation analysis or load testing.

The database

The data set (one record of the database) contains of 4 parts:

1. Bridge description
2. Analytical model description
3. Load testing description
4. Comparison of the load test results and analytical calculations.

A single data base record is allocated to the whole bridge structure or to its part, if the part of the structure exists as an independent static scheme.

The data allocation to sheets (Figure 2) is compatible with parts of the data set:

1. Bridge description
 - 📄 Bridge description sheet
 - 📄 Bridge schemes & photos sheet
2. Analytical model description
 - 📄 Analytical model description sheet
3. Load testing description
 - 📄 Load testing description sheet
 - 📄 Static Loading-Results sheet
 - 📄 Dynamic Loading-Results sheet
4. Comparison of the load test results and analytical calculations
 - 📄 Static Loading-Comparison sheet
 - 📄 Dynamic Loading-Comparison sheet

Figure 2 The application window view – Bridge description sheet is in use and the other sheets are seen as sheet marks.

The bridge description sheet contains general information about the bridge:

- Year the bridge was built (rebuilt),
- The bridge load testing year,
- Bridge design: slab, multi-beam or girder, tee beam, box beam or girders - multiple, box beam or girders - single, frame, orthotropic, truss - deck, truss - thru, arch - deck, arch - thru, suspension, stayed girder, movable, segmental box girder, channel beam, other,
- Bridge structural material: concrete, steel, prestressed concrete/ post-tensioned concrete, wood or timber, masonry, aluminium, wrought iron or cast iron, other,
- Service on bridge: motor road, pedestrian, bicycle, railroad (only in mixed types), other,
- Service under bridge: motor road, pedestrian, bicycle, railroad, waterway, relief for waterway, other,
- Structure length,
- Number of spans,
- Length of spans,
- Deck width,
 - Number of the separate roadways and their width,
 - Number of the separate footways and their width,
 - Number of the separate railroad, width of the railroad & number of the rails,
- Capacity rating - design loads - the short information about capacity rating design loads,
- Dimensions of main carry members - the short information about basic carrying members,
- Condition (in case of old bridges) - the short information about bridge condition,

The Analytical model description includes general information about the analytical calculations method, which was used to calculate the bridge behaviour during load testing:

- Type of the analytical model:
 - Flat, three-dimensional, other,
 - Finite element method, displacement method, other,
- Degree of analytical model: rod, slab, girder, truss, arch, other,

The Load testing description sheet includes general information about the static and dynamic method of loading and the investigation range and measurement methods:

- Type of load testing: testing bridge before putting into service, research testing, assessment of load carrying capacity, other,
- Static loading
 - Method of loading:
 - Loaded heavy goods vehicles: Maximal number of the vehicles (during all loading variants), The average weight of the single vehicle,
 - Other,
 - Table about loading variants and magnitude of the load related to design loads:
 - Description of loaded members and internal forces,
 - Ratio of internal forces caused by test loads related to internal forces caused by design loads,
 - Number of the vehicles,
- Dynamic loading
 - Method of loading:
 - Heavy vehicles running at various constant speeds,
 - Other,
- Investigation range and measurement methods:
 - Static loading: deflections, strains, support displacements, other,
 - Dynamic loading: deflections, accelerations, strains, other.

The Static Loading-Results sheet contains information about ranges of measured quantities for different bridge members during static loading. There are two divisions of measured quantities: first - bridge members loaded directly and bridge members loaded indirectly and second - elastic and permanent quantities.

The Static Loading-Comparison sheet contains information about the comparison factors (set of comparisons) of the measured and calculated quantities. It is one of the most important information for each database record. There is one division of comparison factors: bridge members loaded directly and bridge members loaded indirect.

The Dynamic Loading-Results sheet contains information about the measured bridge characteristics obtained during load testing:

- Bridge behavior under dynamic loads: Range of measured dynamic coefficients related to the method of loading
- Dynamic characteristic of the bridge:
 - Values of the free vibration frequencies,
 - Range of the logarithmic dumping decrements.

The Dynamic Loading-Comparison sheet contains information about the comparison factors of the measured and calculated free vibration frequency.

More information on that issue is available in the chapter 4 of the report.

The database information review

The database includes 110 records with load testing results and analytical analysis from

several countries: Czech Republic, Croatia, Bulgaria, France, Poland, Spain and Slovenia.

The bridge design and material distribution is shown at Figure 3. Because of some complex design type of bridges the total amount of bridges is greater than database records amount.

The database contains many graphics information. The example bridge schemes are presented at Figure 4, the example bridge photos are presented at Figure 5, the additional graphic data to static testing and calculation are presented at Figure 6 and the additional graphic data to dynamic testing and calculation are presented at Figure 7.

There are many analysis of compatibility of test results and analytical calculation. The example presentation of the degree of compatibility in the function of bridge material is shown at Figure 8.

More information on that issue is available in the Appendix A of the report and in the Internet database.

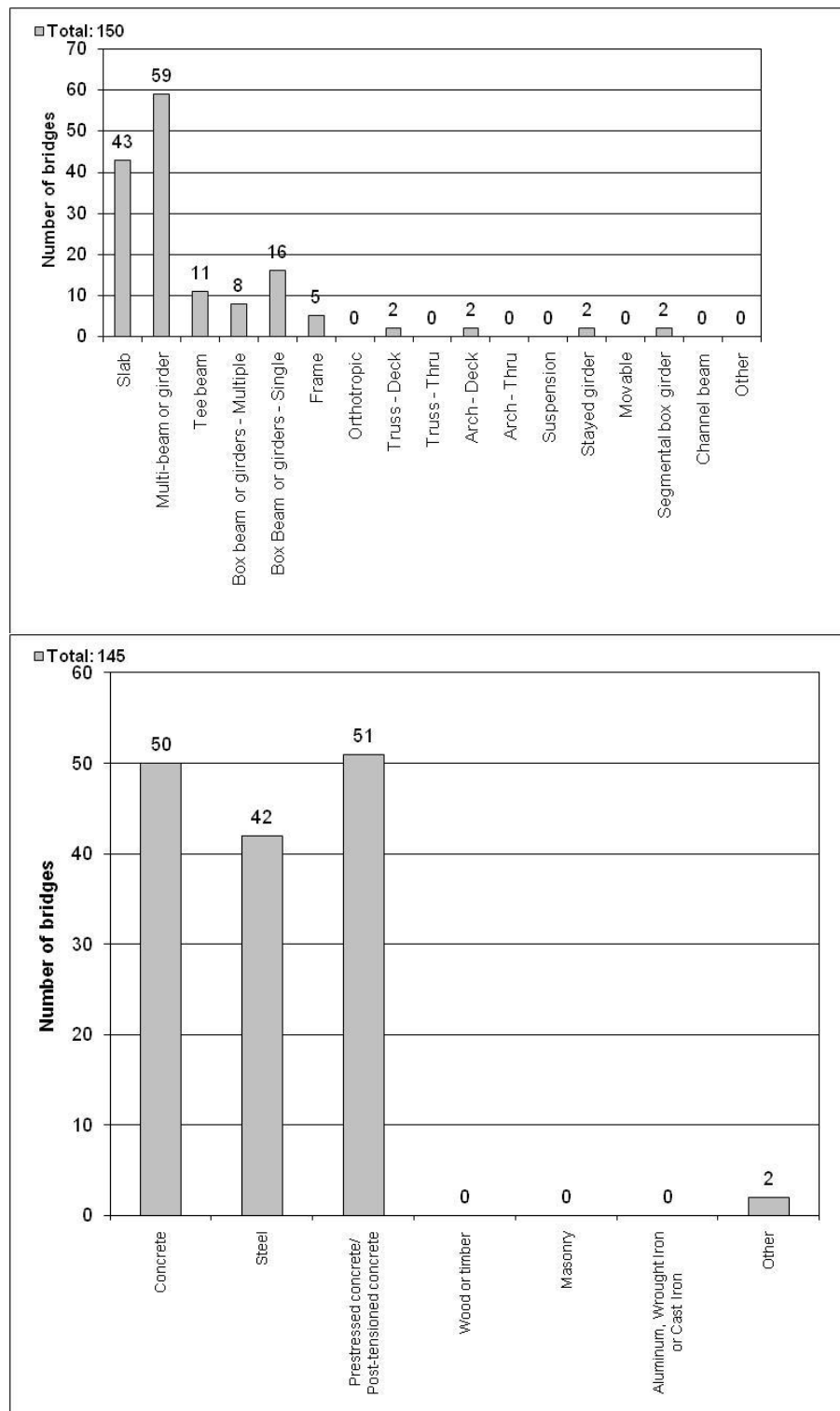
Internet database.

Figure 3 The bridge design (upper) and material distribution (lower) in the database.

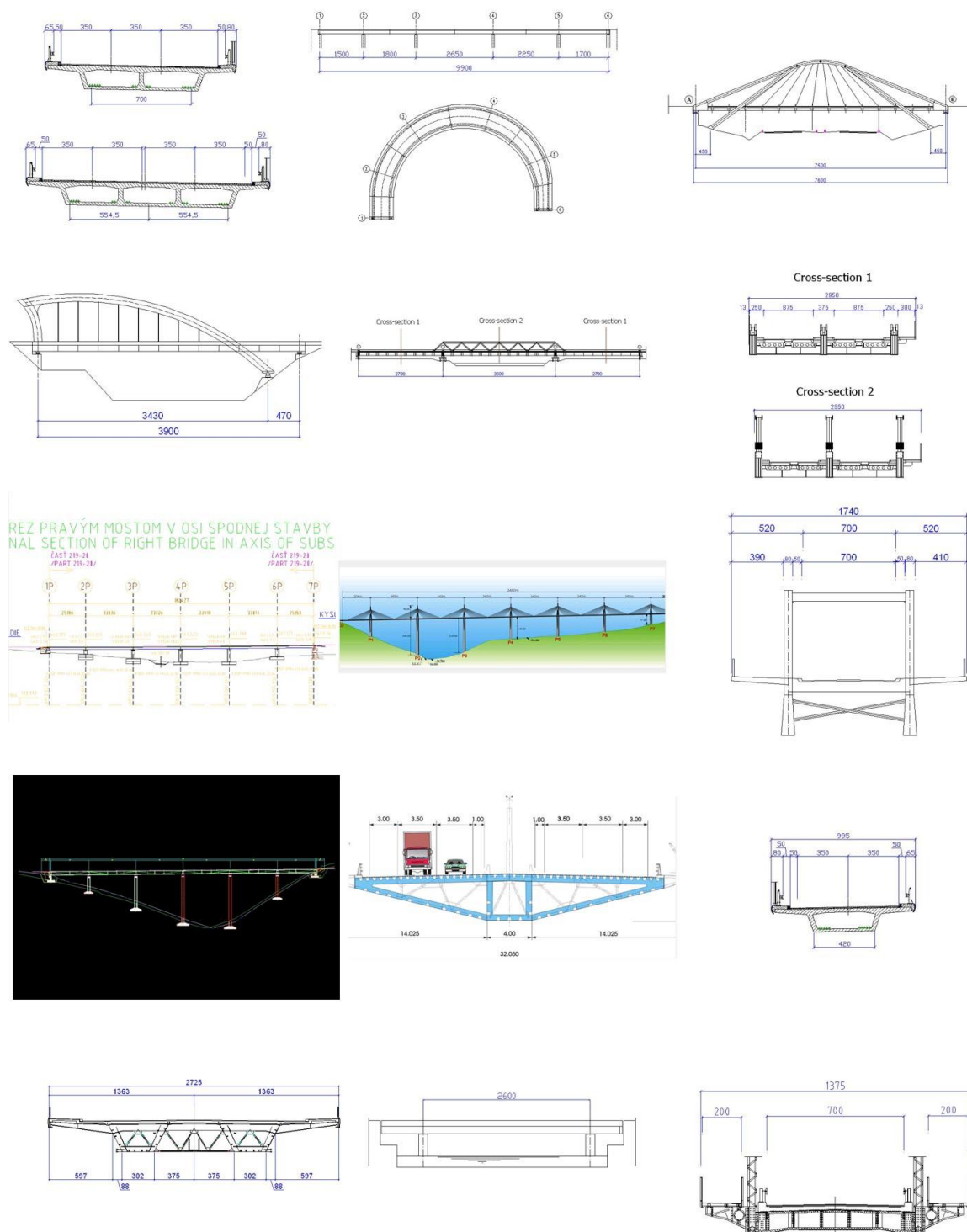


Figure 4 The example bridge schemes.

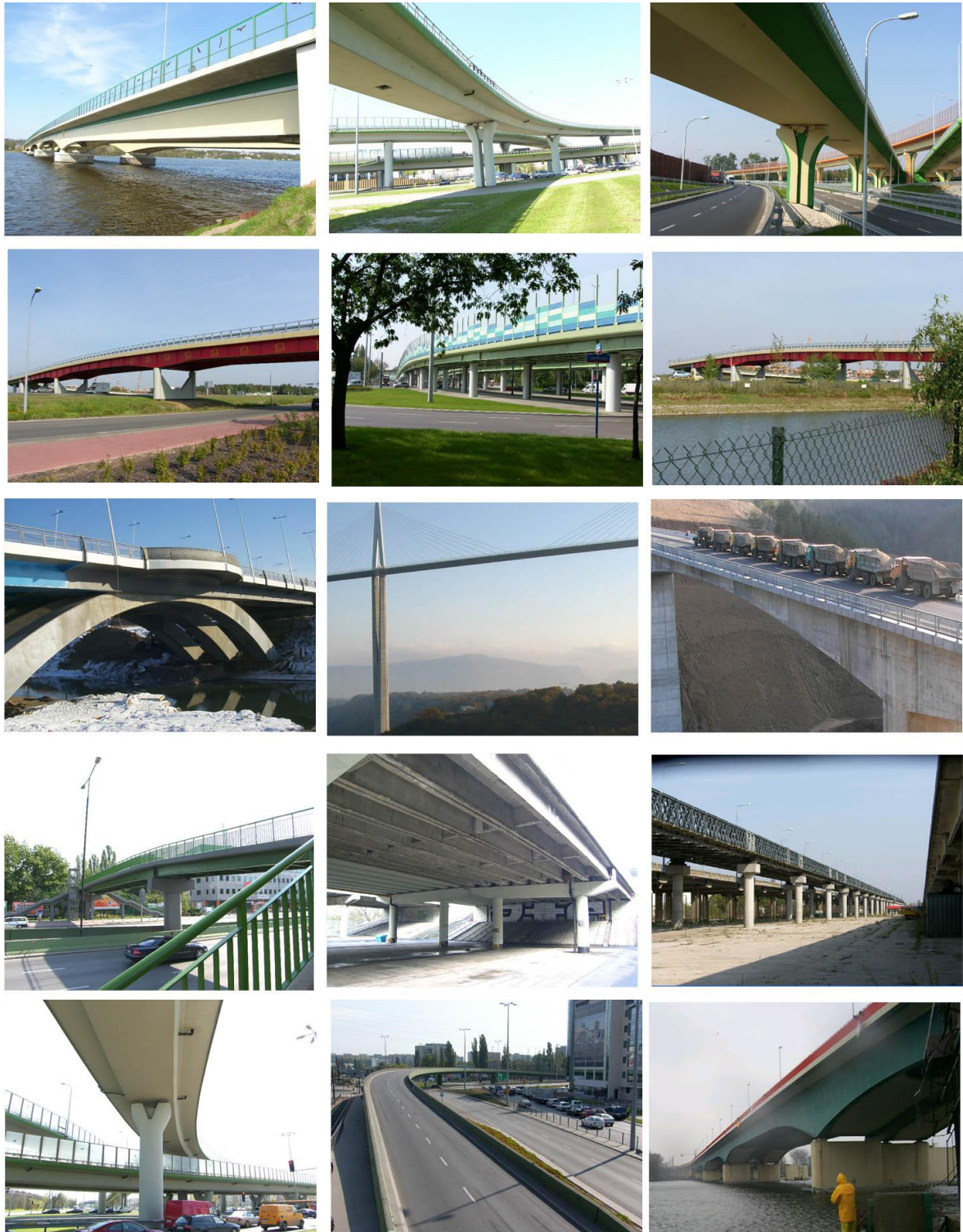


Figure 5 The example bridge photos

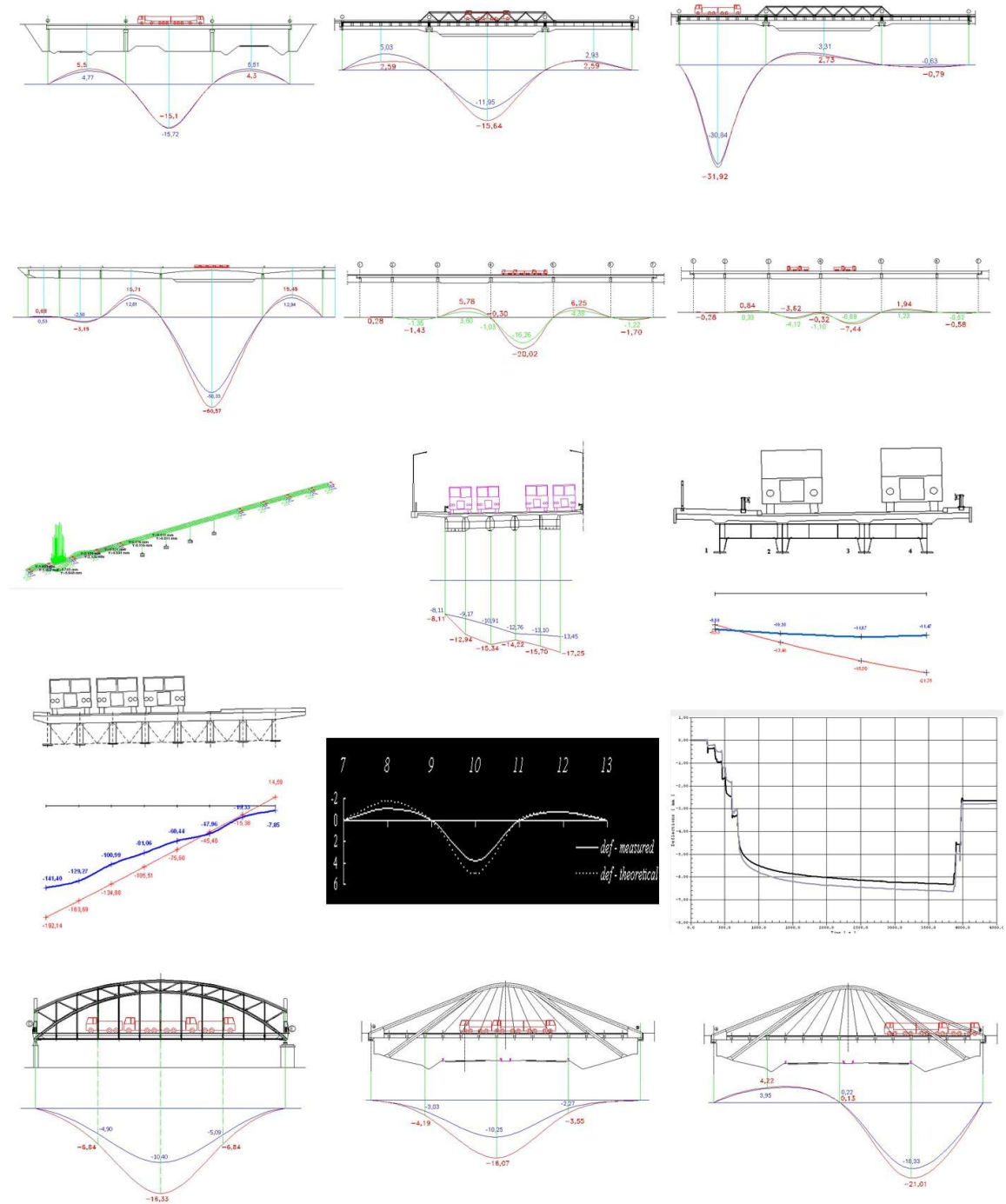


Figure 6 The example additional graphic data to static testing and calculation

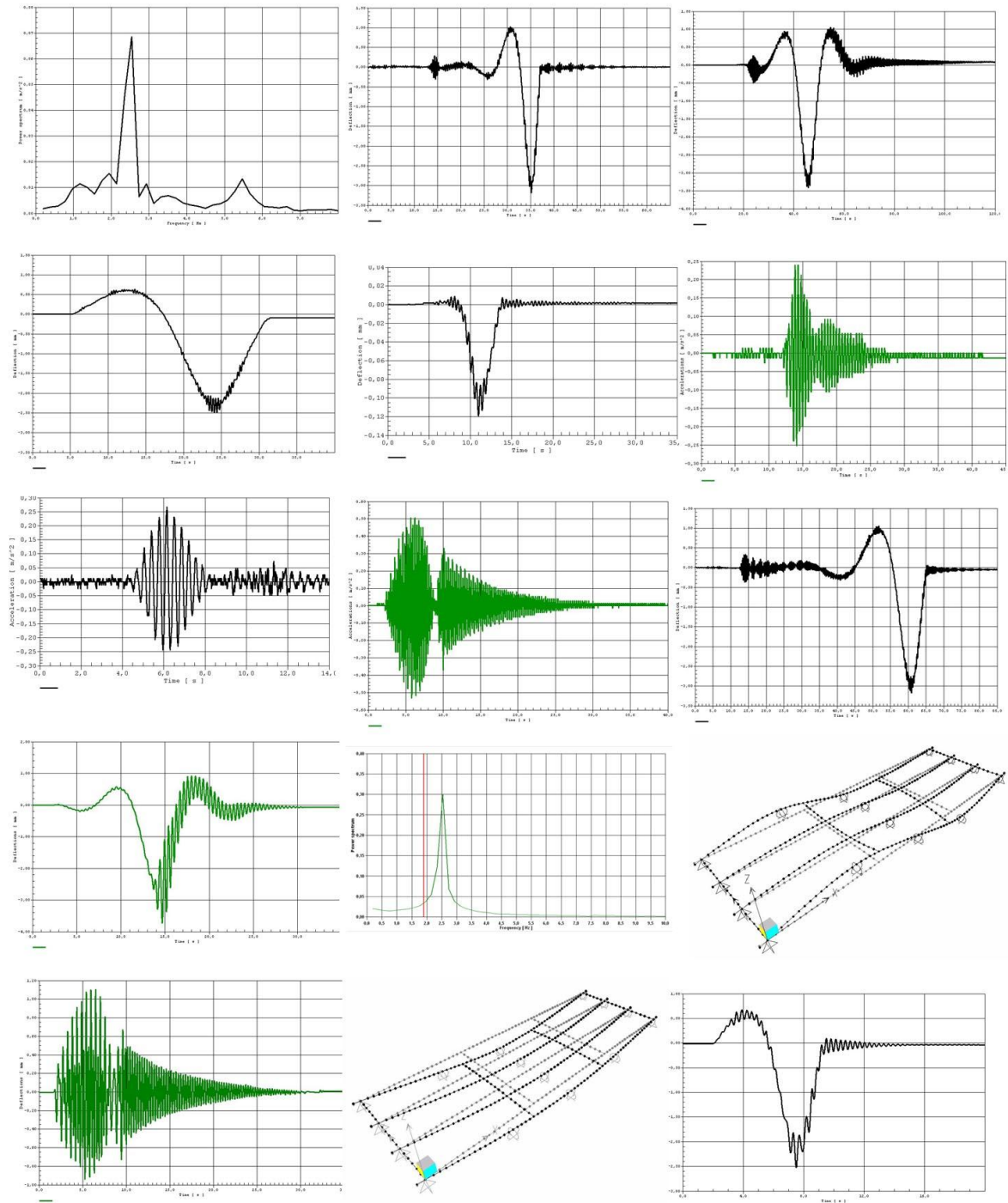


Figure 7 The example additional graphic data to dynamic testing and calculation .

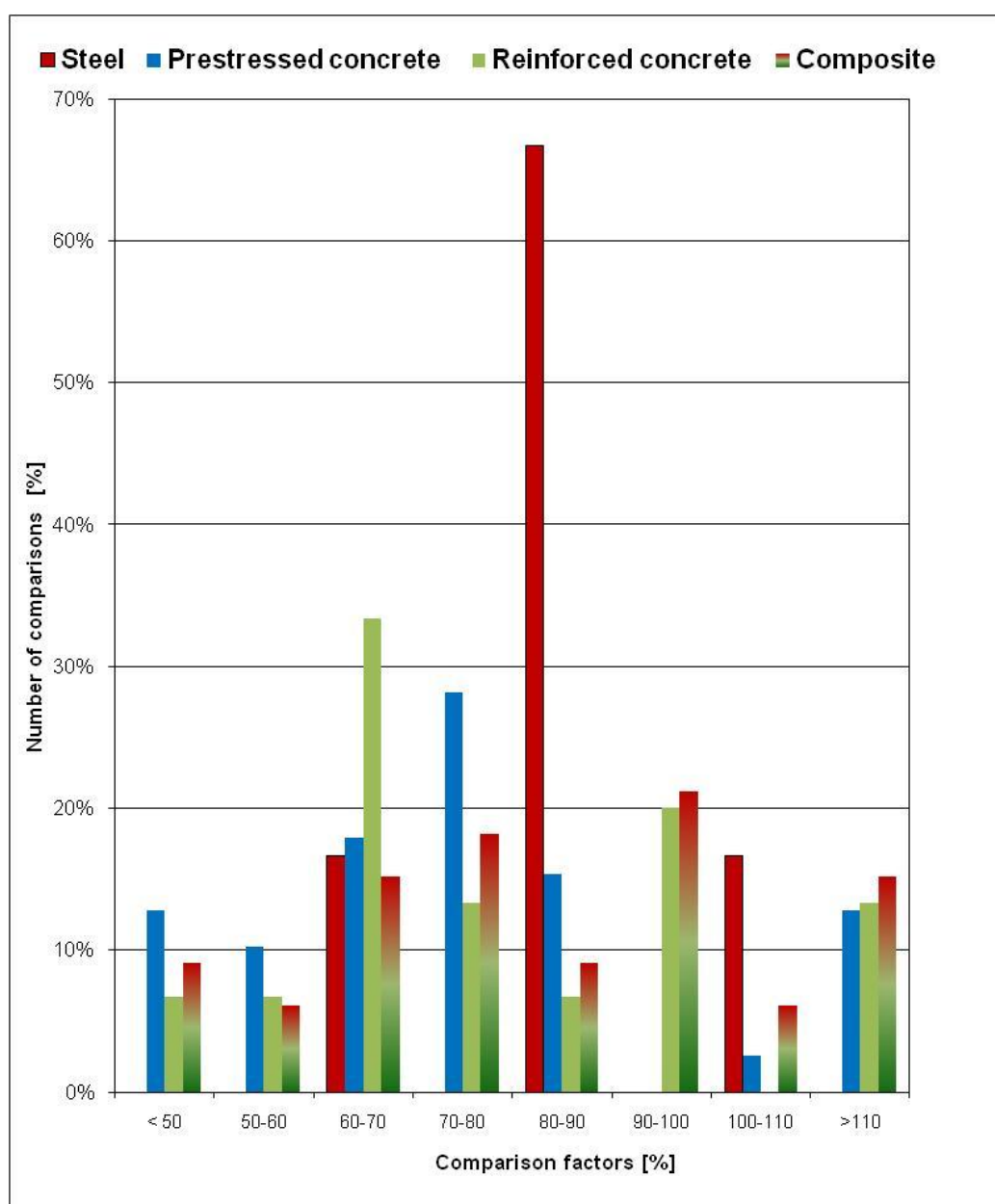


Figure 8 The distribution of comparisons factor in the junction of bridge structural material.

Conclusion

The Internet database of load test results and analytical calculations seems to be useful for the end users, to present quickly the behavior of the structure under the loading and suggest the compatibility range between real bridge behavior and results of the analytical calculations.

The comparison factor review presented in the Appendix A pointed out, that average comparison factors of nearly all bridges have unacceptable match according to the RECOMMENDATION ON THE USE OF SOFT, DIAGNOSTIC AND PROOF LOAD TESTIN. This mean that nearly all analytical models presented in the database requires calibration. The hypothetical assessment of bridge load capacity with the use of those models without calibration would have unacceptable error. Only 3%-36% (depends on the bridge structural material) of presented in the database comparisons contain loaded bridge member with the acceptable match.

The database, to be more useful should contain more information. The additional information about analytical model (the scheme of bridge analytical model, number of the elements and nodes) seems to be very interesting. The suitable for the user would be a full database filling - because of possessed data (by ARCHES Program participants) not all data base fields are filled. The changes require the database developing and inputting the data into the database directly after the load testing execution. At that case the database would be useful for better analytical modeling of bridge structure – calibration based on the similar structure behavior.

3.1.5 Load carrying capacity based on load testing results- Recommendations on the use of soft, diagnostic and proof load testing

Introduction

This summary presents the proposed recommendations concerning the load testing possibilities in bridges with the objective of their safety assessment. The types of tests considered are: soft, diagnostic and proof load test. The recommendations are divided into two parts: 1) recommendations on the most appropriate type of load test according to the proposed objectives of the assessment and 2) recommendations on the use of test results for bridge assessment depending on the type of load test executed.

The analysis and results carried out to support the present recommendations are fully explained in the rest of this document (chapters one to five and corresponding appendices of the D 16 Deliverable). The reader can refer to the background information provided in these chapters to explain the basis of the present recommendations. Information is also provided on soft, diagnostic and proof load tests carried out in 2 bridges selected into ARCHES project to support the present recommendations: the Barcza bridge in Poland (see appendix C of the D 16 Deliverable) and the Gameljne bridge in Slovenia (see appendix D of the D 16 Deliverable)

Recommendations for the selection of test

Three types of load test are feasible in bridges: soft, diagnostic and proof load.

The soft load test uses the actual traffic on the bridge as the loading source. Using a Weigh-In-Motion (WIM) system not only the main characteristics of exciting traffic are obtained, but also information about the structural behaviour of the bridge, through **the calculation of experimental influence lines, load distribution factors and, if measurements are sufficiently long, dynamic amplification factors** to different structural members. In this way, the test is aimed to supplement and check the assumptions and simplifications made in

the theoretical assessment. Therefore, the main objective is to optimise the structural model used for safety assessment. The execution of the test does not require the closure of the bridge to normal traffic. It is also shown in Deliverable how soft load test can be used to evaluate the **characteristic total load effect of traffic action** in a quite simple way, provided sufficient time data of traffic records are available.

Similar to soft load test, diagnostic tests serve to verify and adjust the predictions of an analytical model. However, in this case the level of load in the bridge is higher and introduced by different devices (trucks, water tanks, ballast,...) with accurately measured weight. Normally the bridge is closed to traffic during the execution of the test to better control the relationship between the load level and the bridge response. The loading source may be static or dynamic. The following information can be obtained: **experimental influence lines, dynamic amplification factors, load distribution, dynamic parameters (natural frequencies, mode shapes, damping)**

In the case of proof load test, the aim is not to supplement and check assumptions and simplifications of the theoretical model (as in the case of soft and diagnostic tests), but to provide a complementary assessment methodology to the theoretical one. The aim of the test is not to up-date the parameters of an existing theoretical model, but to discover hidden mechanisms of response that can not appear under “normal” levels of load, but that develop at higher ratios of load and may increase the bridge load capacity. For this reason, in such test, the load introduced in the bridge is relatively high and due to the risks of damaging the structure, this type of tests is restricted to bridges that have failed to pass the most advanced theoretical assessment or when such theoretical assessment is not possible due to the lack of bridge documentation. The objective of this test is to directly obtain the **maximum allowable load in the bridge with a required safety level**.

Based on the main characteristics of the different test types and the resulting data provided by them, the following recommendations were derived:

- 1) Due to its ease of application, the soft load test is particularly useful for:
 1. Old bridges, with no drawings and no information about the design and construction details and about behaviour under loading. Longitudinal and transverse influence lines (distribution factors) can be obtained for normal traffic load.
 2. Posted bridges, to check if the posting (limiting of the traffic loading) is justifiable or it can be released or removed.
 3. Providing input data for efficient management of heavy vehicles with special permits.
 4. To obtain experimental dynamic amplification factors to be used for the assessment of existing bridges under normal traffic
 5. To the assessment of site-specific bridge characteristic total load caused by traffic

2) Candidate bridges for diagnostic load test are those for which an analytical load rating model can be developed. This requires sufficient data and information on as-built bridge details, dimensions and materials or, alternatively, sufficient data obtained through inspection and materials test. Bridges that should be assessed versus dynamic excitations (earthquake, wind,...) are also good candidates for a dynamic diagnostic test.

3) Proof load tests may be performed if documentation is not available and the effects of deterioration and/or damage cannot be evaluated in alternative ways. The use of such tests, due to the risks of collapse or of damaging essential elements of the structure, must be restricted to bridges that have failed to pass the most advanced theoretical assessment and are therefore condemned to be posted, closed to traffic or demolished. If according to the bridge response and material, the failure could be sudden, without warning, proof testing should not

be used.

Recommendations for the quality control of measurements

According to the actual trend to apply quality management systems to many fields of testing activities, it seems also necessary to implement quality system to bridge load testing. Because of the non repeatability nature of bridge testing, it is not advisable and possible to implement a quality system to all process of load testing. In order to guarantee a high metrological quality of the measurements of the realized investigations it would be very useful to cover the most important measurement activities by quality system as:

- deflection
- support displacement
- strain/ stress

The testing programs, structure work analysis, interpretation of testing results and conclusions should not be covered by the quality system.

The quality system should be based on the requirements of the International Standard (EN 2005). The International Standard contains all the requirements that testing and calibration laboratories have to meet to demonstrate that they are technically competent, and they are able to obtain technically valid results. The introduced quality system contains management and technical requirements. The most important management requirements are (EN 2005):

- Having managerial and technical personnel with the authority and resources needed to carry out their duties
- Having arrangements ensuring that its management and personnel are free from any undue internal and external commercial, financial and other pressures and influences that may adversely affect the quality of their work
- Having policies and procedures ensuring the protection of its clients' confidential information and proprietary rights, including procedures for protecting the electronic storage and transmission of results
- Definition of the organization and management structure of the laboratory
- Specification for the responsibility, authority and interrelationship of all personnel who manage, perform or verify work affecting the quality of the tests
- Ensuring adequate supervision of testing and calibration staff, including trainees, by personnel familiar with methods and procedures, purpose of each test and with the assessment of the test result
- Having technical management which has overall responsibility for the technical operation and the provision of the resources needed to ensure the required quality of laboratory operations.

The second set of requirements are the technical requirements connected with the following factors determining the correctness and reliability of the tests:

- Human factor
- Accommodation and environment conditions
- Test and calibration methods and method validation
- Equipment
- Measurement traceability
- The handling of calibration items.

One of the most important reason for which bridge structure tests are covered by a quality system, is the need for receiving valid testing results. Estimation of the measurement

uncertainty for measurements should be based on guidelines (ISO 1993). When estimating the measurement uncertainty of the measurement, it is necessary to take all uncertainty components, which are important in the given situation into account using appropriate methods of analysis. Sources contributing to the uncertainty are not limited to the equipment used. For example, for deflection measurement with transducers it should be complied also the uncertainty connected with gauge installation method.

All equipment used for testing to achieve the required accuracy should be calibrated. Laboratory should have the programme and procedure of equipment calibration. The calibration programme should be prepared in such a way to be capable of measurement traceability.

The limits of measurement uncertainty should be compatible with the expectation analyzing structure behaviour and possibilities to reach on site condition.

Recommendations for the bridge assessment through load testing

Soft loading

At the moment of elaboration of the present recommendations, the soft load testing procedure had only been tested and used on bridges with individual span-lengths shorter than 30 m. The results presented in this document show its applicability also to other bridge types.

When carrying a soft load test, the measurements should acquire at least 100 relevant (loaded) heavy vehicles in each lane are recorded. Typically, 24-hour measurements are performed. This provides a sufficiently high number of results for reliable calculation of influence lines and of load distribution factors.

If the ambition of the soft load test is not only to provide information about structural behaviour but also about the real traffic loading (potentially, for developing the site-specific traffic load model) and/or to evaluate the realistic value of the Dynamic Amplification Factor (DAF), then the measurements should last as long as possible. Depending on the traffic density and probability of appearance of extremely heavy vehicles, measurements should last at least a week for very dense traffic (over 4,000 heavy vehicles on the measured structure per day), or at least 50,000 recorded vehicles during the entire period. If relevant (if traffic patterns change considerably during the year), data should be acquired during the most characteristic period (when the heaviest vehicles are expected). Under any condition, the data sample used for load modeling should contain typical extreme vehicles in the population.

Site specific assessment, either using special assessment loading schemes or site specific traffic load modeling, has considerable potential to prove that bridges which would otherwise have been rehabilitated or replaced are safe. This is because standards are necessarily conservative. Tests in many bridges have shown that dynamic factors reduce as load effect increases. Therefore, a characteristic value of total traffic load (taking into account dynamic vehicle-bridge interaction) to be used in bridge assessment has to take this effect into account. Provided the data is representative, it is shown in the present Deliverable and in Deliverable D10 how soft load testing is an useful tool for calculating the characteristic total load effect. The characteristic total load effect is the nominal value to be used in the assessment to take into account the time period for the assessment (5, 10, 50,... years). The calculation of such total load is done by multiplying the characteristic static load effect by the so-called Assessment Dynamic Ratio (ADR). ADR is different from Dynamic Amplification Factor (DAF) that applies to passages of single trucks, and, therefore, is a more accurate dynamic factor to take into account the actual traffic load within a reference period in bridge assessment. In fact, defining the ADR (Assessment Dynamic Ratio) as the ratio between the total effect and the static effect of traffic load within a defined time period, we can see in

Figure 9 how the variability in ADR narrows as return period increases. The figure corresponds to a simulation of a single-vehicle event in a simply-supported bridge with 25 m span-length. The simulation is done for 100 randomly generated road surface profiles.

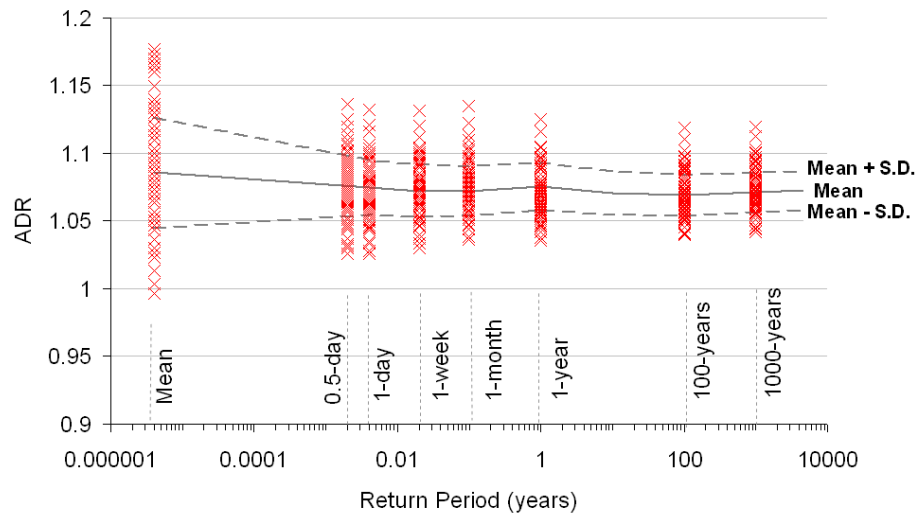


Figure 9. Variation in ADR with return period

In Figure 10 is presented the percentage of error in ADR for several return periods compared to the 1000 year return period. From the figure one may conclude that for a return period of 1-month the percentage error compared to the 1000-year ADR is between -18 % and +24 %, while for a 1-year return period, the percentage varies from -18 % to + 6 % (a positive error results in the conservative assessment of 1000-year ADR).

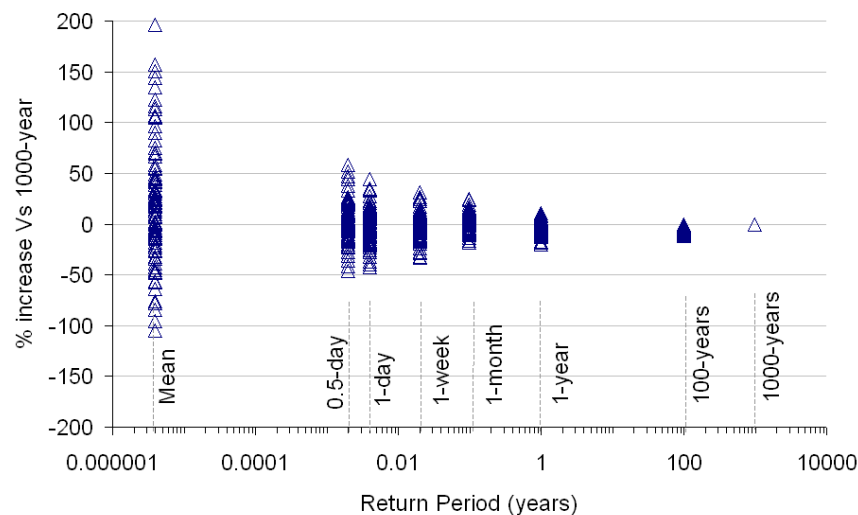


Figure 10. % increase in ADR Vs 1000-year ADR with return period

The reduced variability in ADR with increasing return period can be better visualised in Figure 3 where 1-month ADR is plotted against 1000-year ADR (in the figure each point represents a different road profile). A strong positive correlation between 1-month ADR and

1000-year ADR is evident from the plotted data.

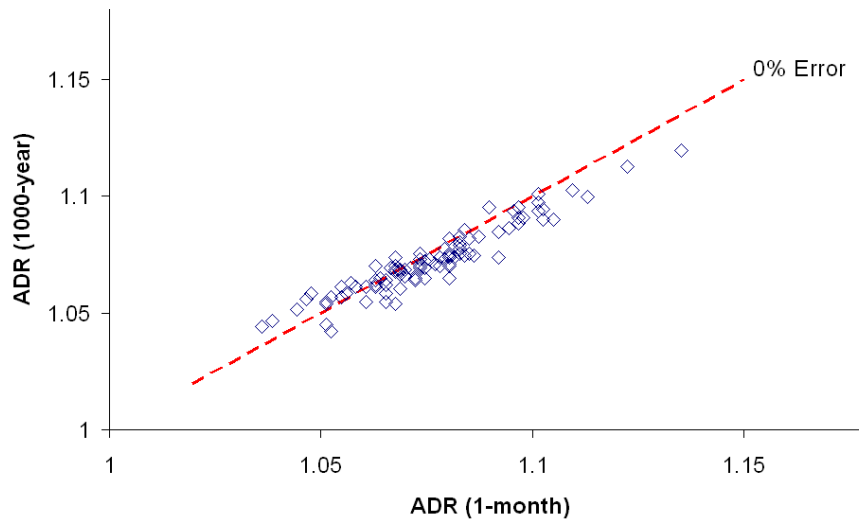


Figure 11. 1-month ADR Vs 1000-year ADR

Similar results are obtained for vehicle-meeting events (see Table 9).

Table 9 Correlation between ADR_R and 1000-year ADR for vehicle-meeting events

	ADR_R						
	Mean	0.5 Day	Day	Week	Month	1-year	100-year
1000-year	0.585	0.750	0.775	0.794	0.841	0.885	0.944

Since it is much more feasible to obtain ADR for a 1-month return period, either by on-site testing through soft load testing or by computational means, it is proposed that by applying an obtained 1-month ADR to the extrapolated 1000-year characteristic static load effect, an accurate assessment of site-specific characteristic total load effect may be made. By comparing the exact distributions of static and total load effect it is seen that there exists the potential for ADR to reduce with return period. This relationship is examined in detail by utilising the exact distributions for static and total load effect and comparing the characteristic values at multiple return periods, for a number of road profiles. It is found that ADR does not necessarily reduce with return period, but rather a more general conclusion is proposed that the variability of ADR reduces with return period. This appears to be the case since it is shown that the ADR from some observed period becomes strongly correlated with the 1000-year ADR as the observed period increases. Furthermore, an acceptable value of 1000-year ADR is efficiently obtained by considering only a 1-month return period, since the ADR for a one month period has a strong positive correlation to the 1000-year ADR. This is verified for both single vehicle events and vehicle meeting events. This finding is particularly relevant for the development of the concept of soft-load testing of bridges, i.e., assessment of a bridge ADR from deformation due to everyday traffic.

The methodology is described in Figure 11. The proposal is to combine independently observed values of site-specific characteristic static load effect (obtained by soft-load testing) and site-specific ADR for a return period of 1 month (also obtained from soft-load tests) to obtain an accurate value for site-specific characteristic total load effect.

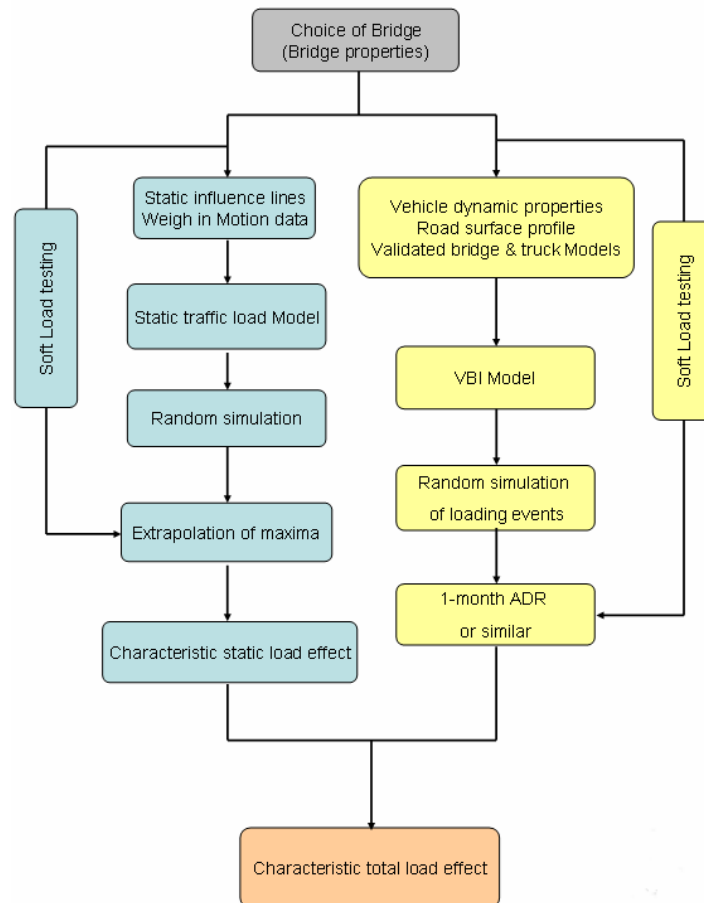


Figure 12 Proposed procedure for assessment of characteristic total load effect based on soft-load testing

The methodology proposed in Figure 12 corresponds to the complete process to obtain the characteristic total load effect. However, a simplified and approximate process is also possible based on a convolution method and the use of experimentally obtained influence lines, load distribution factors and DAF. This alternative method, as presented in deliverable D08, is much easier to implement and is almost as efficient as the true load modelling.

In soft-load tests, in the measurements made in reinforced concrete bridges, special attention must be paid on cracks in the concrete, which often remain hidden to inexperienced personnel. If they are present, which is normal for most RC structures, some strain measurements can differentiate considerably (illogically) from the adjacent ones. The strain level is then either much (typically up to 5 times) higher, if the sensor is put over a crack, or much lower (close to zero), if the sensor is just beside a crack. Figure 13.1 (top) shows an example of measurements where strain sensor 11 was placed over a crack, thus recording abnormally high value of strain. The same figure in the bottom shows similar situation, but with the strain sensor 2 just beside a crack, which resulted in very low level of strain measured by that sensor. If such measurements are not properly calibrated, the calculated bending moment in the critical structural members can be wrong considerably.

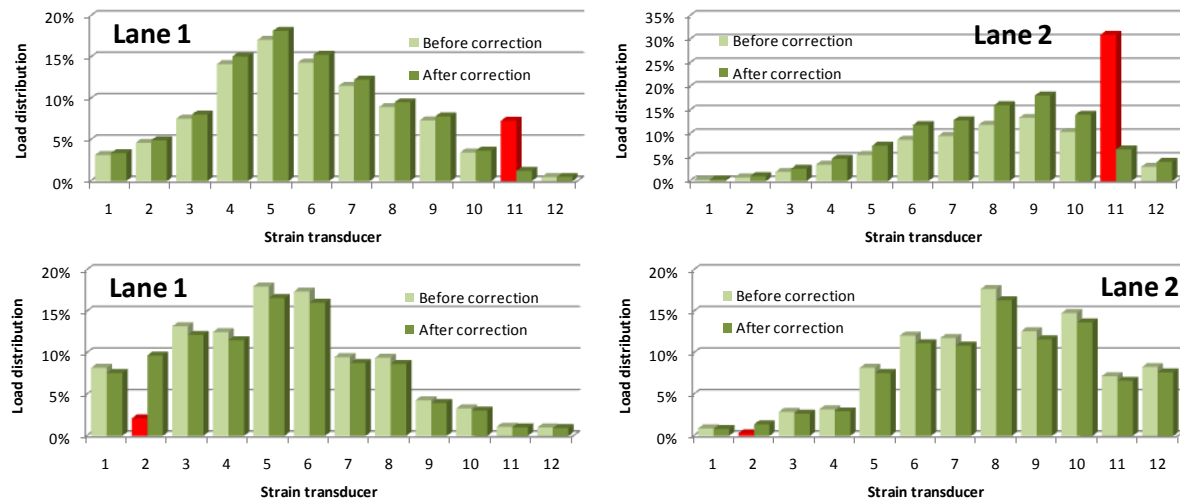


Figure 13. Strain measurements – strain transducer 11 installed over a crack (above) and sensor 2 installed close to a crack in concrete (bottom)

Diagnostic loading

Many countries perform load tests on new and rehabilitated bridges. This extremely useful information should be used to optimize assessment of existing bridges. The Deliverable D07 of ARCHES (Internet Database of Load Test Results and Analytical Calculations) contain information on the correlation between the real structure behaviour (load testing results) with corresponding results of analytical calculation of different types of bridges from different countries (available data from national resources and from other projects, including tests of bridges before putting them into service, assessment of load carrying capacity of existing bridges and load tests done for research purposes). The database of load testing results makes possible to correlate data on load testing with corresponding results of analytical calculation of different types of bridges. The analytical modelling of a structure is characterized by a lot of simplifications and inaccuracies in relation to real structure behaviour. The appendix to the D07 deliverable text description contains setting up of compatibility of analytical modelling with real structure behaviour. The database allows the end users, to judge quickly the behavior of the structure under the loading and suggest the structural assessment method to be used – computation analysis or load testing.

There are two ways to incorporate the results of the diagnostic static tests in the assessment process:

1. By up-dating the structural model and calculation of the new bridge capacity (reliability index, load factor) based on the new model. The idea is to change the bridge properties (area, inertia, modulus of elasticity...) in the way that the theoretical model matches as better as possible the results of the load test.

To this end, an acceptable match is considered to have been reached when the differences between the site-measured maximum deflections and the analytical values are within the following limits:

- +/- 10% for prestressed concrete and metallic bridges
- +/- 15 % for reinforced concrete and composite bridges

Once the model is up-dated, the assessment calculations are carried out using the revised model and it can be used in the recalculation of the bridge safety (reliability index, load factor...)

2. By direct calculation of the load capacity from the test results. In this case, it is assumed that the bridge assessment is carried out using the partial safety factor format and the load

capacity is the value for which the rating live load should be multiplied to reach the failure limit state. The proposed equation is (AAHSTO 2003):

$$LC_T = (1 + K_a K_b) \times LC_C$$

LC_T is the load capacity based on the result of the load test

LC_C is the load capacity based on calculations and before incorporating the results of the load test

K_a can be positive or negative depending on the results of the load tests and is calculated as:

$$K_a = \frac{\varepsilon_c}{\varepsilon_T} - 1$$

ε_T is the maximum member strain measured during the load test

ε_C is the calculated strain due to the test vehicle at its position on the bridge which produced ε_T . It should be calculated using a section factor (area, inertia...), which most closely approximates the member's actual resistance during the test.

K_b is a factor that takes into account the possibility that the bridge has adequate reserve capacity beyond the rating load level and also the load level (compared to the rating load) that the bridge has faced during the test. If the relationship between the un-factored test vehicle effect (T) and the un-factored gross rating load effect (W) is less than 0.4, it is recommended to take $K_b = 0$. If this relationship is higher than 0.7, then a value of 1.0 is recommended if the behaviour of the member during the load test can be extrapolated for a load level of 1.33 W, if not, the value is 0.5.

Proof loading

Two main issues arise when dealing with the application of a proof load test in the assessment of an existing bridge:

1. Which is the maximum load that should be applied to the bridge during the test to guarantee the safety (at a predefined probability level) when the normal daily traffic action will be present in the bridge?
2. How to guarantee that the bridge will not be damaged due to the application of a high percentage of the bridge load capacity during the execution of the test?

The present deliverable presents answers to both questions in proposing the following recommendations:

- 1) A calibration process has been carried out to obtain the so-called proof-load factor, defined as the value that should multiply a nominal value of the traffic action (in the present case the bending moment produced by the live load model in the Eurocode for traffic actions in bridges) to obtain the maximum load effect (maximum bending moment) to be applied to the bridge element in the proof-load test (see 5.3.5 and appendix B). The following variables have been considered in the calibration process:

A.- Safety level: The safety that the test execution should guarantee to the user of the bridge. The safety level is measured through the target values of the reliability index. Three target values were analyzed: 2.3, 3.6, 5.0. The first value corresponds to a nominal probability of failure of 0.01 and is representative of a regular safety level in bridges subject to regular inspections every 2 to 5 years. The second value represents a failure probability of 0.00016 and is normally used in the calibration of design codes (as the Eurocode) taking into account a service life between 50 and 100 years. The last value can be seen as an upper value and

corresponds to a extremely high level of safety (probability of failure equal to 0.00000028).

B.- Span length: 10, 15, 20, 25, 30, 35 m

This covers most of the span lengths in regular bridges encountered in the European highway network.

C.- Bridge type: The longitudinal profile is a simply supported structure. Pre-cast beams and upper slab, massive and voided slab and box-girder are considered for the cross-section

This covers most of the encountered cross-sections in concrete bridges for the span lengths accounted for. The parameter considered is the ratio between the effects (bending moment at mid-span) of the permanent load (G) and the traffic load (Q) as defined in the Eurocode of actions on highway bridges.

D.- Traffic action: Five traffic scenarios have been considered, one representative of Western Europe (The Netherlands) and other representative of New Member States and Central and Eastern Europe (Czech Republic, Poland, Slovakia and Slovenia). Therefore, 5 different country-specific traffics were studied.

The main characteristics of the traffic composition (histogram of gross vehicle weights, axle load and distances between axles) are shown in Appendix A of the D 16 Deliverable. A summary of the traffic data is presented in Table 10. All sites have 2 measured lanes. In all countries, except in Slovakia, the 2 lanes are in the same direction. More information on the recorded traffic data is available in appendix A of the D 16 Deliverable and Deliverable D08. According to the results reported in Deliverable D10 that show how the dynamic amplification due to traffic decreases with the total load applied in the bridge, leading to a value closer to 1.0 for extreme load events, and taking into account that proof load testing is related to the ULS of resistance (high level of traffic load in the bridge), then, in the calibration process a dynamic amplification factor equal to 1.0 has been considered.

Table 10 Main characteristic of traffic data used in the calibration

	Netherlands (NL)	Slovakia (SK)	Czech Republic (CZ)	Slovenia (SI)	Poland (PL)
Directions	1	2	1	1	1
Total trucks	646,548	748,338	729,929	147,752	429,680
Time span in weeks	20	83	51	8	22
Number of weekdays with full record	77	290	148	39	87
Trucks per day lane 1	6,545	1,031	4,490	3,158	3,708
Trucks per day lane 2	557	1,168	261	135	314
Trucks per day (both lanes)	7,102	2,199	4,751	3,293	4,022

To take into account the site-to-site variability of the traffic action, in the present report the analysis and calculation of proof load factors have been done with a coefficient of variation of the traffic effect of 20 %. In the deliverable is also presented a methodology to be used in the case that particular traffic data from a specific bridge site would be available. This is of relevant importance in the case that the bridge would be located in a highway with significant lower Average Daily Truck Traffic (ADTT) than the one observed in the WIM stations used in the calibration (see Table 10). The simplified method proposed in this deliverable can be used to derive a more accurate COV for the bridge site and a reliability analysis executed to define a more accurate proof load factor for the specific bridge.

E.- Permanent additional load: The additional dead load that may appear in the bridge after

the execution of the proof load. This additional permanent load normally reflects the increment of the pavement thickness due to repaving.

F.- Existing bridge documentation: Two cases are considered: the existence or not of bridge documentation and information (drawings, materials specifications,...) to calculate the nominal value of the resistance and dead load at the time of test execution. In the case that the nominal value of the resistance is unknown due to the lack of documentation, the assumption made is that the load level reached in the test execution is precisely this nominal value.

The envelope of results for proof load factors obtained for the representative traffics of the NMS (Czech Republic, Poland, Slovenia and Slovakia) are presented in tables 3 to 5 for documented bridges and in table 6 for non-documented bridges. The proof load factor has been normalized to the Eurocode traffic action. This means that the proof load factor represents the number of times that the bending moment caused by the live load model defined in the Eurocode should be applied to the bridge during the proof load test. If the bridge supports this load level during the test without any damage indication, then the safety to the passage of normal traffic is guaranteed with a defined reliability level. It should be understood that normal traffic refers to the legal vehicles and therefore the values can be applied only to the bridges located in highway sites where a reasonable enforcement and apparent control of overloads is present. It also assumes that exceptional heavy vehicles run properly escorted.

In the Table 11 to Table 13, R is the actual resistance of the bridge calculated with the available data for geometry and material's properties and the design code. R_n is defined as:

$$R_n = \gamma_D G_n + \gamma_L Q_n$$

with G_n , Q_n = nominal value of permanent and traffic load, and γ_D , γ_L the partial safety factors for permanent and traffic action (1.35 and 1.50 respectively). The nominal value of traffic action, Q_n , is according to the Eurocode 1.

The specific values of proof load factors obtained for each country are detailed in the text of the Deliverable D16 (see chapter 5).

Table 11 Documented bridges. Proof load factors of concrete bridges in bending proposed for NMS as function of actual resistance and span-length (Reliability index $\beta = 2.3$)

R/R_n	Span-length (m)					
	10	15	20	25	30	35
1.0						0.31
0.9	0.15	0.28	0.45	0.55	0.59	0.61
0.8	0.51	0.58	0.69	0.78	0.82	0.84
0.7	0.63	0.69	0.82	0.94	0.96	0.98
0.6	0.72	0.78	0.92	1.00	1.04	1.05
0.5	0.78	0.84	0.96	1.04	1.07	1.09

Table 12 Documented bridges. Proof load factors of concrete bridges in bending proposed for NMS as function of actual resistance and span-length (Reliability index $\beta = 3.6$)

R/R_n	Span-length (m)					
	10	15	20	25	30	35
1.0	0.71	0.78	0.94	1.08	1.16	1.17
0.9	0.82	0.89	1.10	1.23	1.28	1.29

0.8	0.92	1.01	1.21	1.32	1.36	1.37
0.7	1.01	1.10	1.28	1.37	1.40	1.41
0.6	1.07	1.14	1.30	1.39	1.43	1.44
0.5	1.10	1.17	1.32	1.41	1.45	1.45

Table 13 Documented bridges. Proof load factors of concrete bridges in bending proposed for NMS as function of actual resistance and span-length (Reliability index $\beta = 5.0$)

R/Rn	Span-length (m)					
	10	15	20	25	30	35
1.0	1.37	1.47	1.71	1.85	1.89	1.89
0.9	1.44	1.53	1.77	1.89	1.93	1.93
0.8	1.49	1.57	1.79	1.91	1.95	1.95
0.7	1.52	1.60	1.81	1.93	1.97	1.97
0.6	1.54	1.62	1.82	1.94	1.98	1.98
0.5	1.55	1.63	1.82	1.95	1.99	1.99

Table 14 Non-documented bridges. Proposed proof load factors in NMS and CEEC countries.

Span length (m)	Reliability Index (β)		
	2.3	3.6	5.0
10	0.83	1.13	1.57
15	0.89	1.20	1.65
20	1.01	1.36	1.85
25	1.08	1.44	1.97
30	1.11	1.46	2.00
35	1.12	1.48	2.01

The values presented in Table 11 to Table 14 are applicable to the case of concrete bridges in bending. A similar procedure as presented in chapter 5 can be applied to derive similar values to be used with other materials or other failure modes (shear, torsion,...).

Alternatively to the use of the values presented in Table 11 to Table 14, the possibility is to make a specific calculation of the proof load factor for a particular bridge. In fact, in the project a simplified methodology is derived to statistically define (mean value and COV (Coefficient of Variation)) the random maximum traffic action in the bridge. Using WIM techniques it is easy to obtain the following parameters for the actual traffic on the bridge: distance between axles and axle loads for the most representative truck in the bridge, histogram of weights of representative truck, weights of the representative truck for 1000 year and 1 week return period. With all this data, the random variable that models the traffic action is statistically characterized (the mean and COV of a Gumbel distribution are obtained) and, then, a specific reliability analysis is feasible to define the target proof load according to the site-specific traffic conditions, as described in 5.3.5 of the D 16 Deliverable.

In 5.3.6 of the D 16 Deliverable is presented an example of application of the calculation of the target proof load.

- 2) Acoustic emission (AE) has been identified as a useful technique in the follow up of the loading process in proof load tests in order to stop the load increase before any damage can be inflicted to the bridge. In fact, the results from the tests on real bridges carried out

in ZAG laboratory in Slovenia (see Deliverable D08 and Appendix D of the D 16 Deliverable) and Barcza bridge in Poland (see appendix C of the D 16 Deliverable) have shown that monitoring with acoustic emission sensors made possible to evaluate the cracking limits of the concrete members and to stop the load increase without introducing any damage to the bridge. Thanks to the AE signals was possible to evaluate the cracking limits without introducing any significant damage to the girders. The simple follow up of the deflection-load diagram or strain-load diagram as incremental loading is introduced in the bridge, stopping the test when some sign of non-linearity is detected, does not guarantee the possibility of not creating any damage to the bridge (see figure 6 and full information in appendix C of the D 16 Deliverable). In fact, in the case of Barcza bridge even after the detection of the cracking by visual inspection, the load-deflection diagram continued to be linear and no sign of change in the slope was detected.

In Deliverable D08 are presented the main aspects (theoretical background, sensor technology, data analysis,...) of the acoustic emission technology. The interested reader is referred there for more information on the technique.

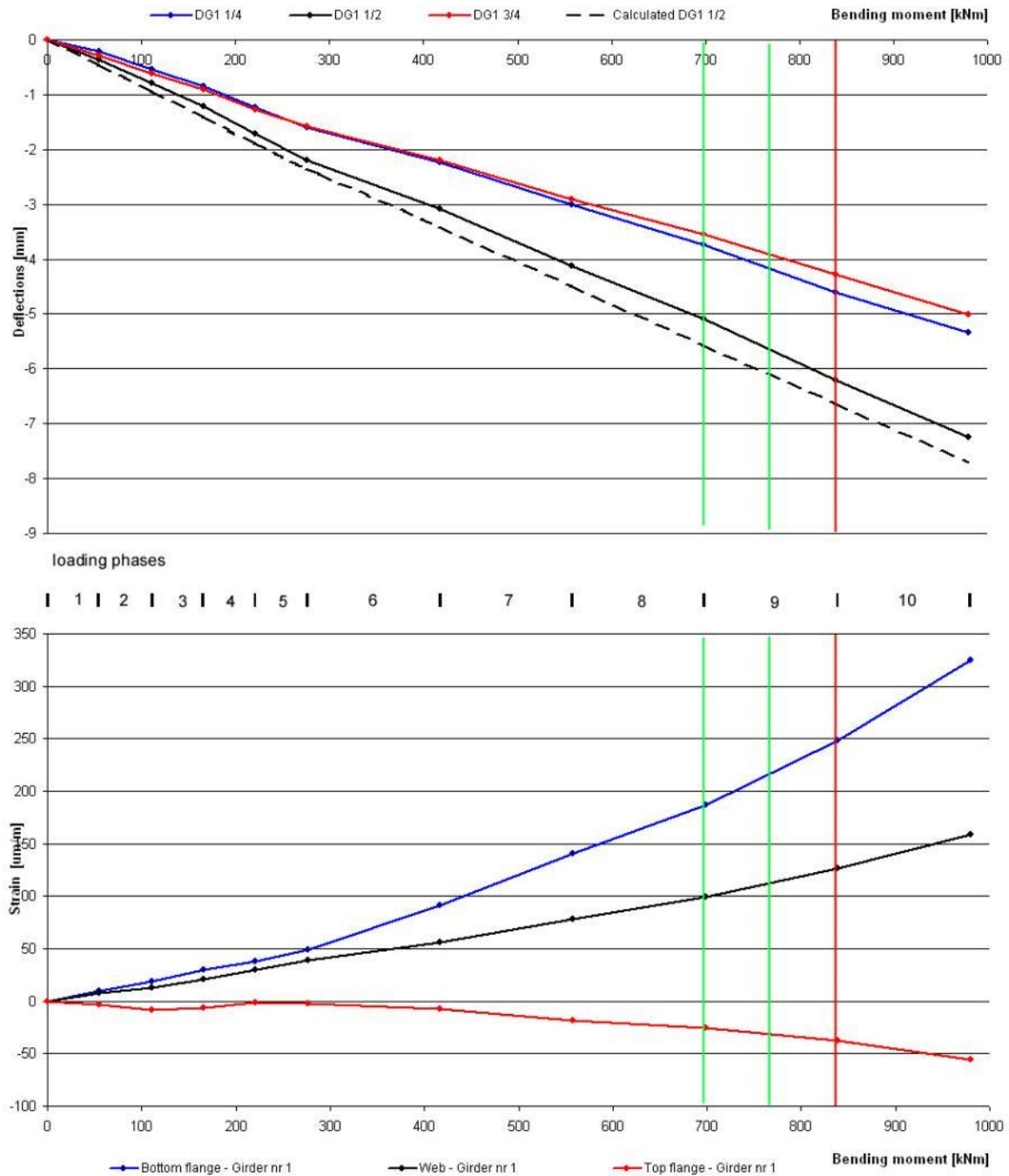


Figure 14 Results of measurements and theoretical analysis in Barcza bridge. Upper: deflection (measured and calculated) - bending moment diagram of girder no 1; Lower: strain-- bending moment diagram in girder no 1. The green vertical lines mark the loading level where load testing should have been stopped (directly before macro-cracking appearance) on the base of the AE results. The red vertical line marks the loading level where the cracking was detected by visual inspection.

3.1.6 *Reducing dynamic loading of bridges - Recommendations on dynamic amplification allowance*

Introduction

The various approaches to assess bridge traffic load (statically) have been described in Deliverable D08. The motives behind the accurate assessment of bridge lifetime load have been discussed, in terms of increases in heavy trafficking and variations in vehicle populations at a European level. The methods of collection of data used to develop bridge traffic load models have also been addressed. Finally, Deliverable D08 has examined the determination of those combinations of heavy vehicles contributing to bridge lifetime static load for different Weigh-In-Motion (WIM) sites. However, the dynamic allowance to consider for the bridge lifetime total load remains to be established.

So, Deliverable D10 describes the developments in the definition and implementation of a more accurate dynamic allowance for traffic loading on bridges. For this purpose, two concepts of dynamic allowance are employed: Dynamic Amplification Factor (DAF) and Assessment Dynamic Ratio (ADR). DAF is defined here as the ratio of the maximum total load effect to the maximum static load effect caused by the passage of the vehicle or vehicles over a bridge. In the latter, both total and static load effects refer to the same traffic loading event and to the same section in the bridge. ADR is the factor that multiplied by the characteristic static load effect will provide the characteristic total loading effect for a given return period. The characteristic total loading effect and the characteristic static loading effect do not necessarily correspond to the same traffic event.

During ARCHES, theoretical simulations and site measurements have been carried out to provide a more accurate determination of the dynamic factors due to traffic loading. The main sections of this investigation can be divided in the following groups:

- Review of current practise on dynamic allowance in bridge codes and standards.
- Recommendations on how to obtain a site-specific ADR using numerical simulations. Guidelines are provided on how to build a vehicle-bridge interaction (VBI) finite element model and how to obtain an ADR for a given road profile, traffic population and return period.
- Recommendations on how to obtain a site-specific ADR using bridge measurements due to traffic. Total and static strains have been measured using the Bridge Weigh-In-Motion System, Si-WIM, on a number of bridge sites and recommendations on dynamic allowance have been provided for each of these sites.
- Other topics reported in ARCHES: (a) Quantification of the differences between the section holding the largest total bending moment and the midspan section in a simply supported structure; (b) Evaluation of the dynamics associated to critical static loading scenarios as in Deliverable D08, i.e., exceptionally heavy vehicles such as cranes, and comparison to the dynamics of typical European 5-axle articulated trucks; and (c) extension of the analysis to situations with a bump prior to the bridge, deteriorated bridges or pre-existing vibrations prior to bridge loading.
- General recommendations on dynamic allowance based on bridge length and road class. These recommendations are based on the analysis of the bridge response due to a large amount of traffic-crossing scenarios where vehicle properties and road profile have been widely varied using Monte-Carlo simulation.

The structure of the deliverable is summarised in the scheme of Figure 15. Therefore, the document is accompanied by appendices on mathematical models employed for structural models of bridges and vehicles, the simulation of their dynamic interaction, and a review of

the influence of vehicle, bridge and road profile parameters on dynamic amplification.

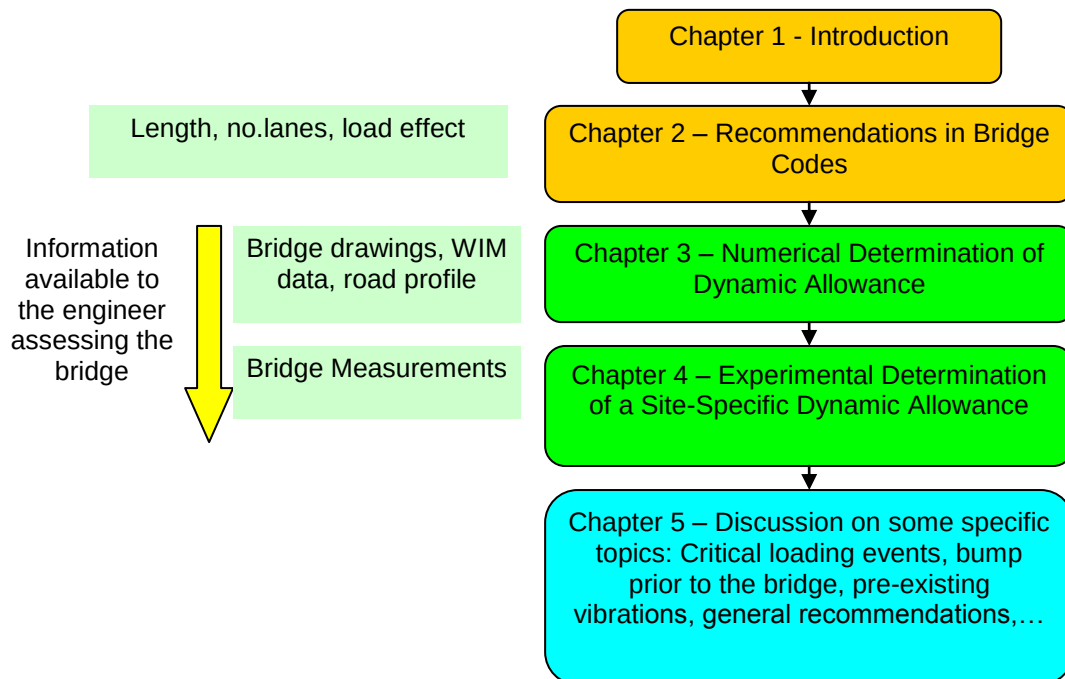


Figure 15 Structure of D10 – Dynamic Allowance

Current practice in dynamic allowance

When assessing a bridge, it is important to take into count the bridge code that was in practise at the time the bridge was designed. Chapter 2 of the Deliverable D 10 reviews the practice in dynamic allowance for many countries and periods of time. If there was no site-specific information available to the engineer, these recommendations represent conservative values to follow. It is common practice to use a DAF or a similar parameter to allow for the uncertainties associated with the structure, the material and the applied load. A more realistic characterisation of the total load effect would require experimental testing and/or the use of complex computer models. The current Eurocode traffic load model is based on the statistical combination of static traffic load effects and DAFs. The latter have been derived from numerical simulations and the average values for global effects are shown in Figure 16 for one, two and four loaded lanes.

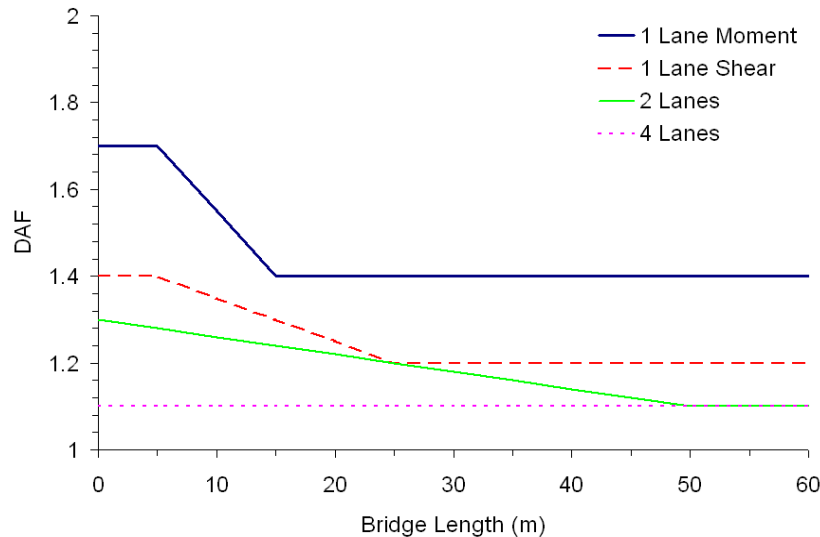


Figure 16. Average Global Dynamic Factors

The Eurocode values above are necessarily conservative to cover for an entire range of bridges with different mechanical characteristics, boundary conditions, and the large number of uncertainties associated to the vehicle-bridge interaction problem. The sections that follow describe how this uncertainty can be reduced by gathering knowledge on the bridge response to the traffic imposed to it, and how to obtain a more realistic dynamic allowance.

Dynamic amplification factor for characteristic static load effects

Chapter 3 explains how to numerically determine a dynamic amplification factor when some bridge, traffic and road characteristics are known. Appendix A contains a review on how these characteristics influence the overall response of the bridge. The assessment of traffic loading on bridges is subject to large levels of uncertainty. While some allowance is provided in design codes for variable traffic conditions, they are conservative to allow for generalisation at a safe level. A further level of conservatism occurs due to the independent manner in which critical static load and the corresponding allowance for DAF are specified. In particular, investigations in this Chapter and in Section 5.6 show that certain bridges are not susceptible to high levels of vehicle-bridge interaction when loaded by a ‘critically’ heavy vehicle or a ‘critical’ combination of vehicles. Chapter 3 presents the results of a range of numerical studies into the site-specific level of total load effect (dynamic + static) and corresponding allowance for dynamics for typical medium span highway bridges. The method proposed herein allows for a more accurate assessment of lifetime total load effect on a specific bridge.

The vehicle models utilised in the VBI finite element simulations can be broadly classified using two distinct groupings; rigid bodied vehicles, and articulated vehicles. Chapter 3 and Appendix B contain full descriptions of each type of vehicle in terms of the mathematical modeling of their suspension and tyre characteristics, axle-spacing and distribution of Gross Vehicle Weight (GVW) through individual axles. The standard models, as shown in Figure 17(a) can be easily modified to represent alternative vehicle configurations by removing/inserting extra axles and/or modifying axle configuration. The procedure to characterise the dynamics associated to the critical loading cases governing the traffic

assessment for a specified return period is demonstrated for a 32 m long simply supported with two-lanes of traffic running in opposite direction. The bridge is of the beam-and-slab type with 5 longitudinal concrete beams and 5 transverse diaphragms as shown in Figure 17(b).

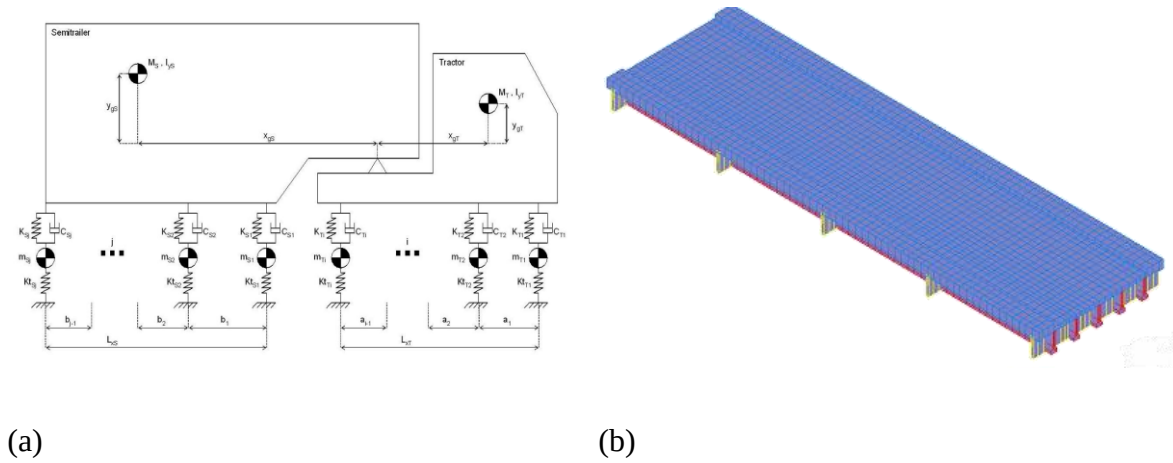


Figure 17. Finite Element Models: (a) Elevation of Articulated Vehicle Sprung Model; (b) Bridge

For bridge assessment purposes, the characteristic static load effect can be found using conventional extrapolation methods: maximum static load effect per day is measured or simulated; the data are fitted to an Extreme Value distribution and extrapolated to find the characteristic static value (more details can be found in Deliverable D08). Figure 18 shows the plotted monthly maxima for a typical year. In the figure, the 10 monthly maximum load events for each type of load scenario (1-truck, 2-truck, etc.) are presented. From the observed years of monthly maxima the 10 most critical events overall are extracted (10 worst monthly maxima). As expected for a bridge of this type and span length the majority of critical events are 1-truck and 2-truck events with occasional 3-truck events contributing.

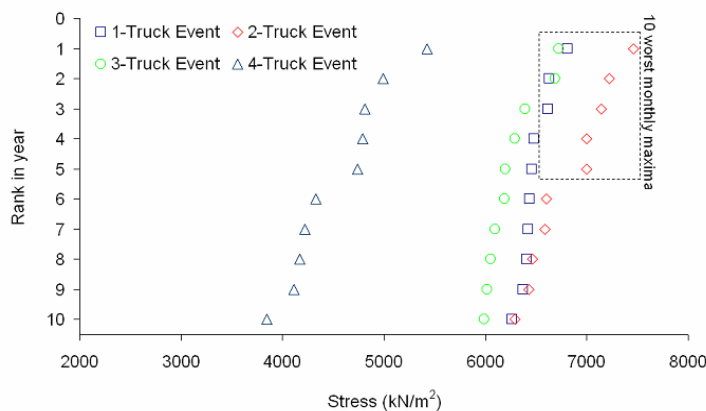


Figure 18. Determination of Worst Monthly Maxima based on Weigh-In-Motion Data for bridge in Figure 3(b)

In Figure 19, the total stress and the DAF associated to the worst 100 static loading cases

resulting from Monte-Carlo simulations are plotted in the figure. The mean DAF of these 100 critical loading events is 1.035 with a standard deviation of 0.041. Two relevant conclusions can be extracted from the figure: (1) the variability of the DAF associated to critical loading events is small and (2) the difference between the worst static loading event (ranked 1) and the static loading event ranked 100 is so large that the probability of finding a traffic event outside these top 100 events causing a larger stress becomes negligible. The Eurocode traffic load model has an implicit in-built DAF of 1.17 (Figure 16), but in this case, the analysis of the traffic on the site for a particular 10-year return period has led to an ADR of 1.06 (ratio of maximum total to maximum static).

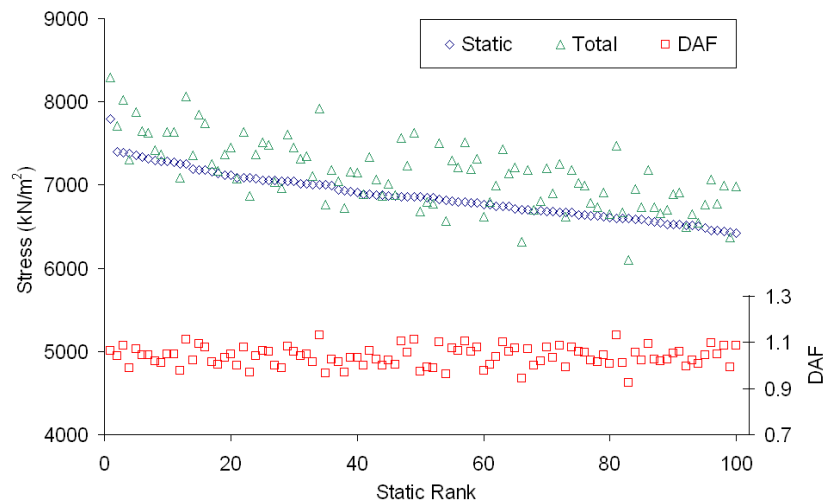


Figure 19. Total Stress and DAF versus Worst 100 Static Stress Loading Cases

Experimental determination of dynamic allowance

Chapter 4 shows how bridge measurements can be used to experimentally obtain a site-specific dynamic allowance. First, the validity of the approach is numerically tested with simulations. Based on the results of these simulations, the chapter proposes a method to calculate an ADR based on strain field measurements. The capability of the Si-WIM system to measure maximum total strain and estimate the maximum static strain for each traffic event is then used to provide a site-specific recommendation for ADR that can be used in bridge assessment. Three of the sites under investigation are defined in the table that follows:

Table 15. - Site Characteristics

Site	Bridge Length	Type	No. of Events	Period. of Measurements	No. of Days
The Netherlands	7.3 m	Integral	52694	3 rd -17 th December '07	15
Trebnje	8 m	Integral	50937	14 th May '07 – 16 th June '07	34
Vransko	24.8m	Simply Supported	147524	25 th Sept. '06 – 21 st Nov. '06	58

Theoretical Justification

The dynamic allowance associated to the critical loading cases analysed in Chapter 3 are clearly lower than the one associated to light vehicles. This is experimentally verified with measurements of DAF and vehicle weights on site. Recent advances in Bridge Weigh-In-Motion technology allow to measure not only vehicle characteristics and weighs from measured strain under the bridge soffit, but also to provide a DAF value for each vehicle event using filtering techniques. Figure 20 shows the relationship between DAF and maximum static strain for two different bridge sites. Values in the Eurocode (Figure 15) would suggest a higher DAF for both bridges (particularly, the shorter Trebnje bridge), but evidence show that for the heaviest vehicles, the maximum DAF does not exceed 1.1 (represented by a horizontal dotted line in Figure 20). The smaller scatter of the tail associated to the maximum static strains of these experimental figures appear to resemble the theoretical results for critical loading cases of Figure 19.

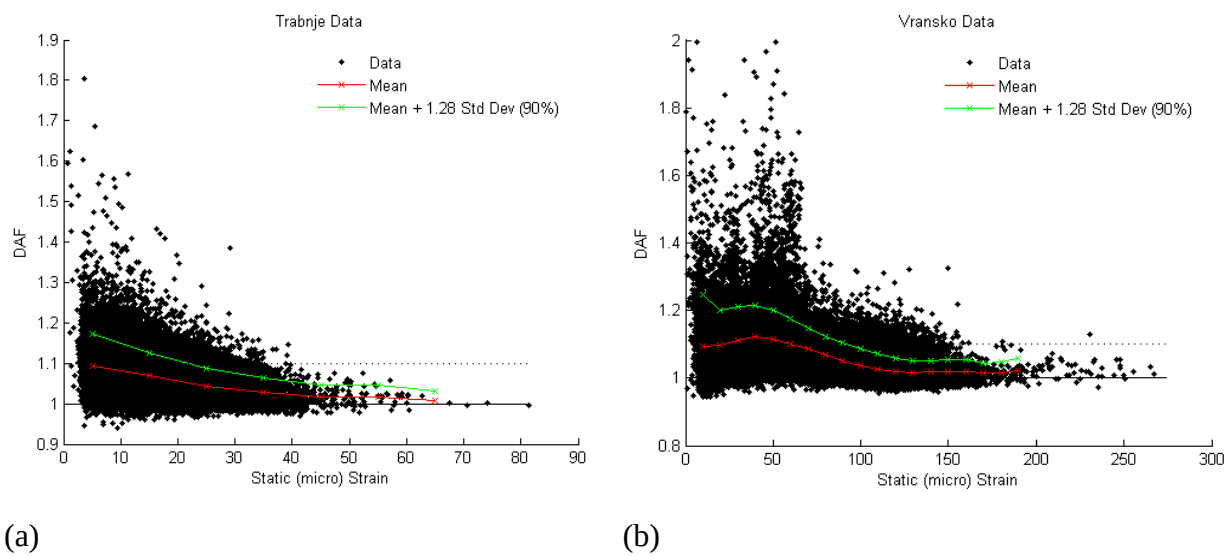


Figure 20. Measured DAF versus Static Strain: (a) Trebnje, (b) Vransko Bridge

The response of a bridge is simulated using a one-dimensional numerical vehicle-bridge interaction model consisting of a single 5-axle vehicle of variables GVW and velocity that traverse a 25 m long simply supported beam. The probability distributions of each variable are defined by WIM data (Figure 21(a)), and are used to obtain the characteristic value for static load effect. Figure 21(b) shows the result of plotting the static bending moment resulting from sampling the passage of traffic over the bridge from the GVW distribution in Figure 21(a). The similarities between measurements from Figure 20 and numerical results from Figure 21(b) are clear; typically, the higher the static strain gets, the smaller the DAF and the variability of DAF become.

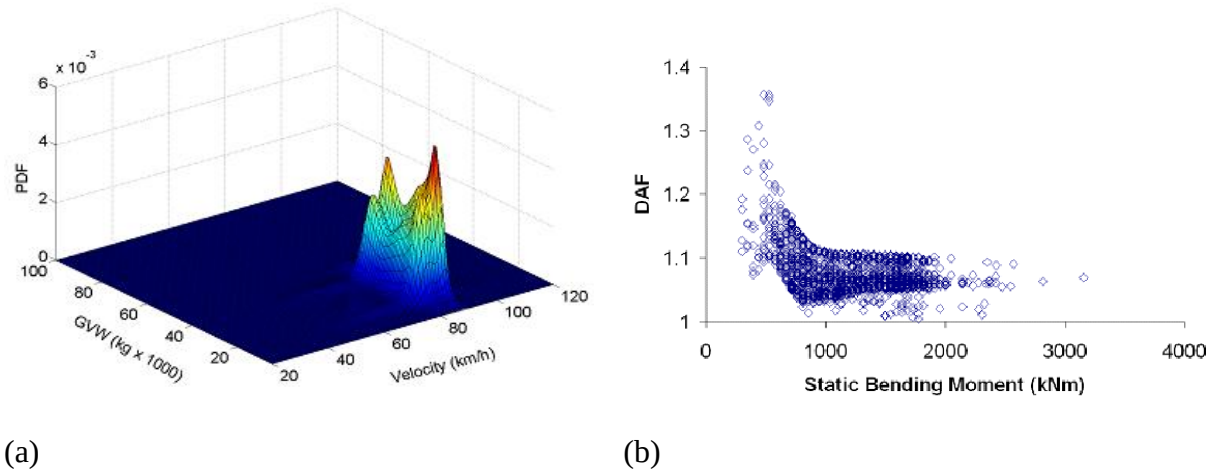


Figure 21. (a) Probability Density Function of GVW and Velocity; (b) DAF versus Static Bending Moment

The characteristic total load effect (including VBI) is required for assessment purposes in this theoretical exercise. It is of interest that these two characteristic values (and hence, its ratio, ADR) may not necessarily arise from the same loading scenario. Similarly, but independently, the distribution of characteristic total load effect is obtained. Then, the cumulative distribution function of maximum static and total bending moments can be generated as shown in Figure 22(a). Comparison between the total and the static results yields the site-specific allowance for dynamic interaction and a given return period; an ADR of 1.06 results for the 1000-year return period sought in this numerical example (Figure 22(b)).

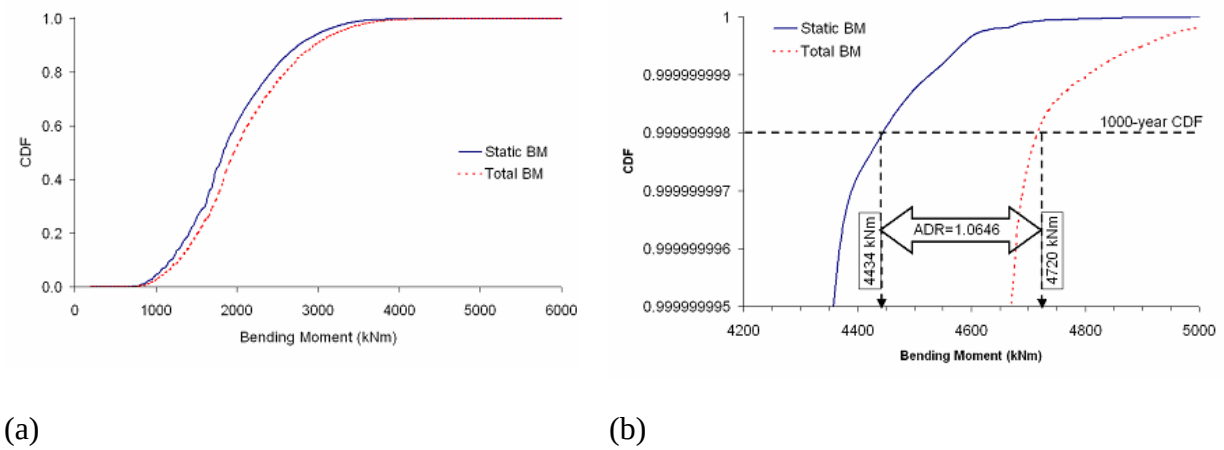


Figure 22. (a) Cumulative Distribution Function for Maximum Static and Total Bending Moment; (b) Determination of Assessment Dynamic Ratio

The ADR value found above faces the problem that in practise it is not possible to take measurements for the return period of the structure. The effect of return period on ADR is analysed in Chapter 4, and illustrated in Figure 23. It can be seen that a relatively short period of time can be used to give a close estimate of ADR. This is a consequence of previous findings such as the fact that longer measurement periods will lead to traffic events causing larger static responses, and larger static responses are typically associated to smaller % of dynamics (Figure 19, Figure 20 and Figure 21(b)).

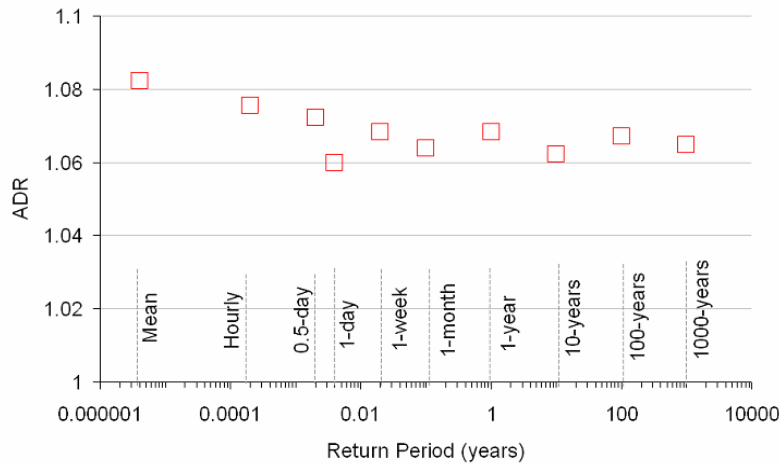


Figure 23. Variation of Assessment Dynamic Ratio with Return Period

A further theoretical investigation consisting of a two-truck meeting event, typical of critical loading scenarios, is carried out with an increased number of design variables. In this study both trucks have different GVW and velocity values, with the meeting location of the vehicles also varied. Additionally, 100 different road profiles within class A ('very good' according to ISO) are analysed. These are the 100 points appearing along a vertical line for different return periods in the horizontal axis of Figure 24. It can be seen how as the return period increases, the influence of the road profile (or variability of ADR with the profile) decreases.

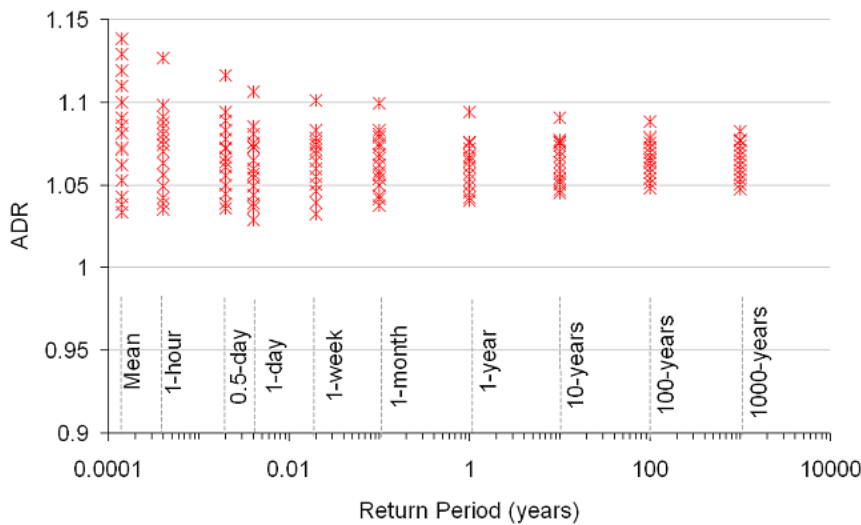


Figure 24. Variation of Assessment Dynamic Ratio with Return Period and Road Profile

Experimental Implementation

The dynamic allowance associated to the critical loading cases analysed in Chapter 3 are clearly lower than the one associated to light vehicles. This is experimentally validated with bridge measurements shown in Figure 20. A vast amount of dynamic measurements were taken during the ARCHES project. 2-axle and 5-axle trucks were the dominant truck classes and a sample of their DAF distribution is illustrated in Figure 25 for the three bridge sites

described in Table 15. The higher modes represent the heaviest subclasses within a given truck configuration. The figure below is found in agreement with previous investigations that generally address: (1) Larger DAFs are associated to lighter vehicles (lower modes) and (2) Larger DAFs are associated to vehicles with smaller number of axles.

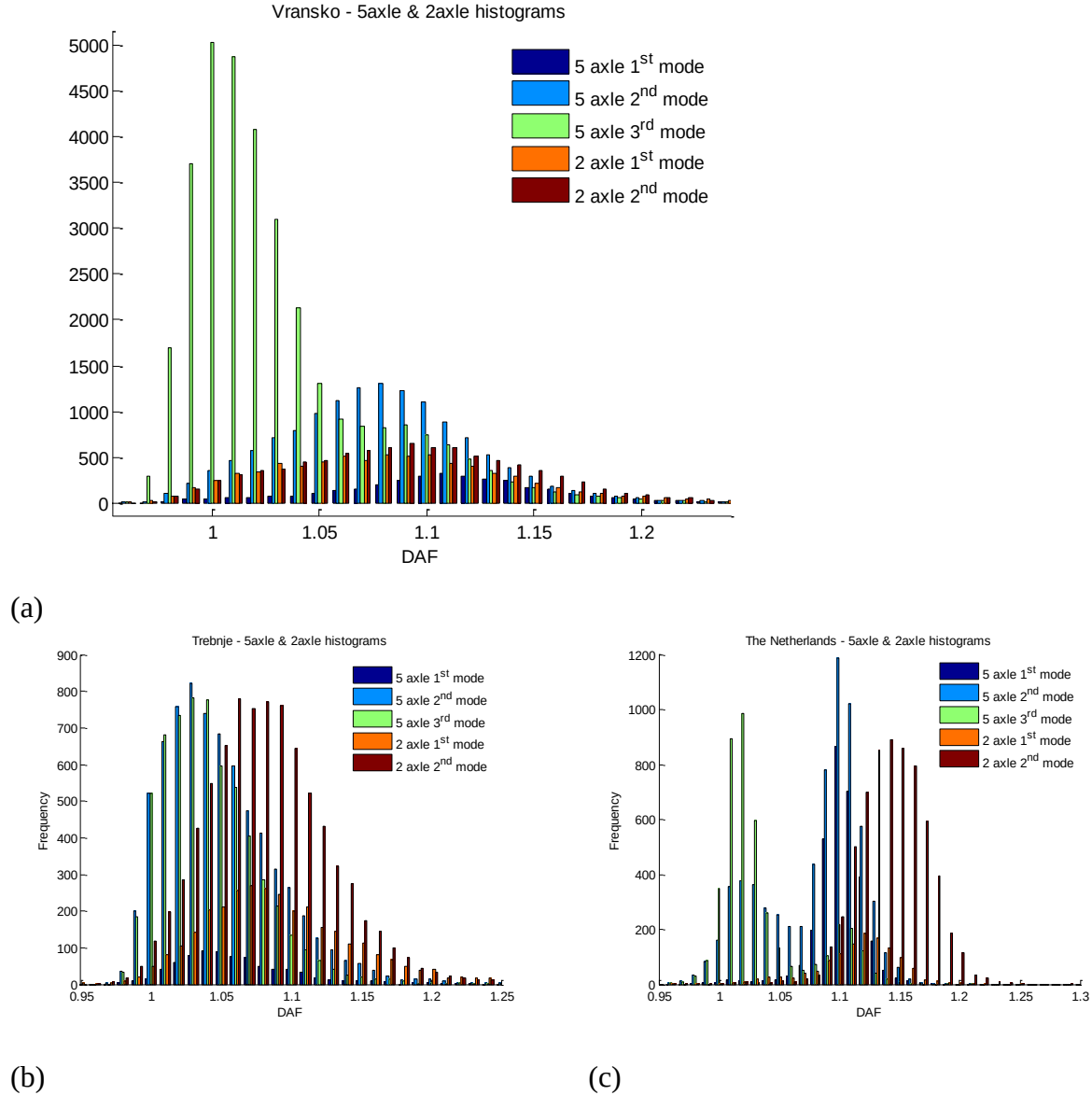


Figure 25. Distribution of DAF by Vehicle Subclasses: (a) Vransko; (b) Trebnje and (c) The Netherlands

Figure 26(a) shows the variation of ADR for different vehicle subclasses in the Vransko bridge (i.e., the ratio of worst possible total load effect divided by worst possible static load effect for a given vehicle subclass and measured number of trucks). There is a clear trend for ADR and the variability of ADR to decrease as the sample size increases (as found in theoretical simulations, i.e., Figure 23), except for boundary errors appearing at the extremes (these could be due to outliers, vehicles changing lanes or some kind of interference that corrupted the measurements). As expected, ADR appears to be smaller for the heaviest vehicle subclass (5th axle, 3rd mode). Figure 26(b) compares the ADR of the 3rd mode of the 5-axle vehicle for three different sites. If boundary errors were ignored, once the sample was

large enough -as for the Vransko bridge-, the ADR does not oscillate as much and tends towards a lower bound value. Finally, Figure 26(c) compares the ADR of the three sites when considering the full data set (all vehicles). From this graph, an ADR of 1.05 can be recommended for the Vransko bridge. It can be seen that Figure 26(c) exhibits more oscillations than Figure 26(b), since the sample of heaviest vehicles is reduced and not as representative as for the 5-axle vehicle class. If the duration of the measurements is limited, it may not be possible to gather enough information on the dynamic amplification associated to the critical loading cases causing larger strains (denoted by higher oscillations). However, there are vehicle classes such as the 5-axle articulated truck traffic event which occur frequently and their dynamic behaviour can be characterised accurately. For the three sites, it appears that the heavy 5-axle vehicle class provides a conservative estimation of what the ADR associated to the heaviest critical loading cases may be. Sections 5.5 and 5.6 of the Deliverable provide further evidence that the dynamics associated to critical static loading cases is smaller than in the case of typical 5-axle European trucks.

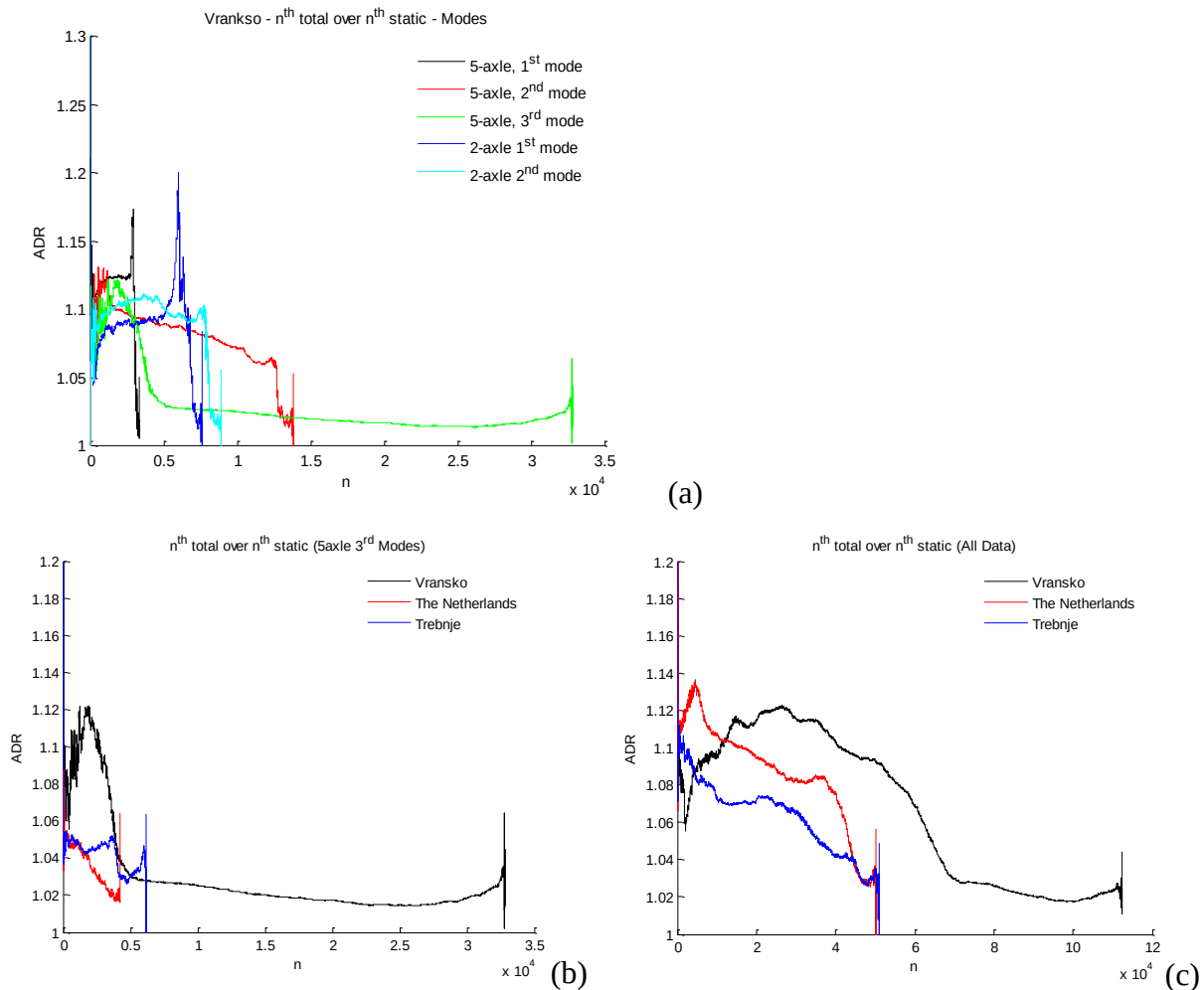


Figure 26. Variation of ADR with Measurement Period for: (a) 2-axle and 5-axle Vehicle Subclasses in Vransko Bridge; (b) Heaviest 5-axle Vehicles in 3 Bridge Sites, and; (c) Full Data Set in 3 Bridge Sites

Measurements of Dynamic Amplification factors

The new generation of Bridge Weigh-In-Motion (B-WIM) systems, which use instrumented bridges from the road network to weigh heavy vehicles, enables measurements of DAF of all vehicles (or loading events with several vehicles) that cross the bridge.

Several new algorithms to automatically calculate DAF were tested within the ARCHES project. Due to the sensitivity of accuracy of results of WIM measurements it was decided not to use the indirect method of reconstructing the static strain response from WIM results, but rather to condition the measured strain signals by low-pass filtering to remove the dynamic component. These procedures were tested on the Vransko bridge. The most reliable results were obtained (with far the lowest number of outliers that required further verifications) when the averaged FFT spectra were applied. It was shown then the shape of the spectrum converges rapidly after averaging a few tens of the loading events, caused either by the individual vehicles or by the multiple-vehicle events.

This is demonstrated in Figure 27, which displays the averaged spectra of one hour and one day of traffic responses on Vransko bridge. The two are practically identical. From such spectra, the cut-off frequency is determined that is then used in the real-time DAF calculation.



Figure 27. The averaged spectra after one hour and one day on Vransko bridge

DAF measurements results

Figure 28 presents as an example the DAF factors obtained on the Vransko bridge as a function of the induced static strain. Each dot in the graph represents one loading event with at least one vehicle heavier than 35 kN. These are divided into single vehicle events in both lanes (yellow and green diamonds), events with meeting one vehicle above and one below 35 kN (purple squares) and events with 2 vehicles exceeding 35 kN (blue circles). These values are compared against the thresholds (orange flat line) taken from the pre-Eurocode Slovene bridge design code ($DAF = 1.19$) and the of the Guideline for Reliability Based Classification of the Load Carrying Capacity of Existing Bridges published by the Danish Roads Directorate in 2004. For the extreme loading events this curve closely follows (envelops) the experimental DAF values.

Figure 29 summarises results from the four of five DAF sites. Average values and standard deviations of all measured DAF values were calculated for 20 different strain levels in a way that result at certain strain level comprises DAF values of all vehicles that induced *static strain* above this value. This gives a good indication how average DAF factors change for heavier loading events. Results for all sites clearly converge towards 1.

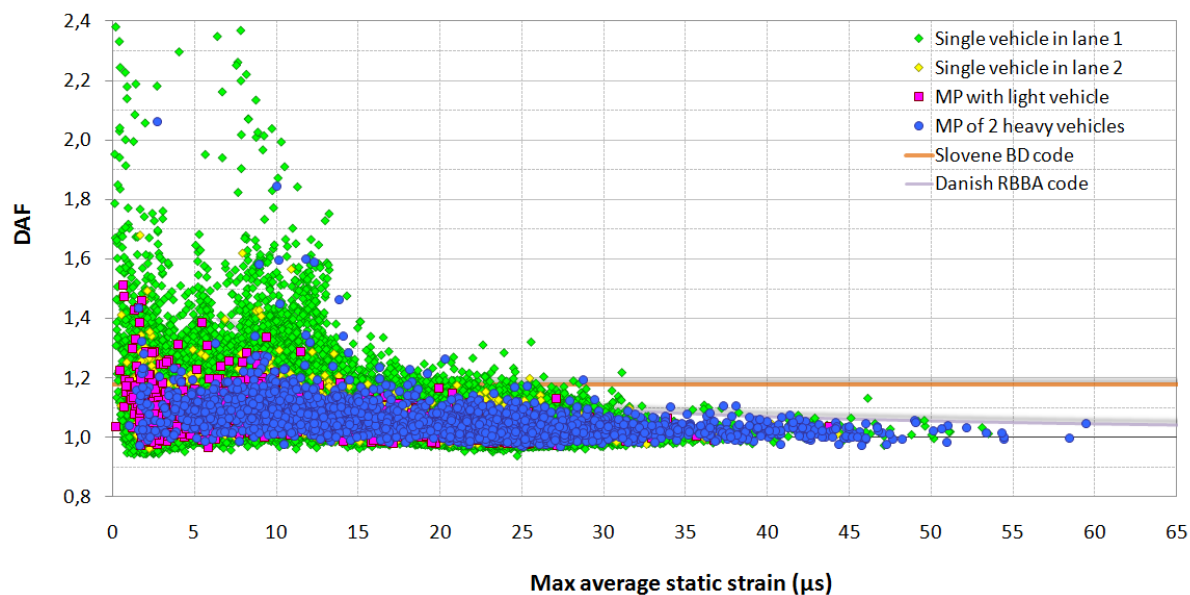
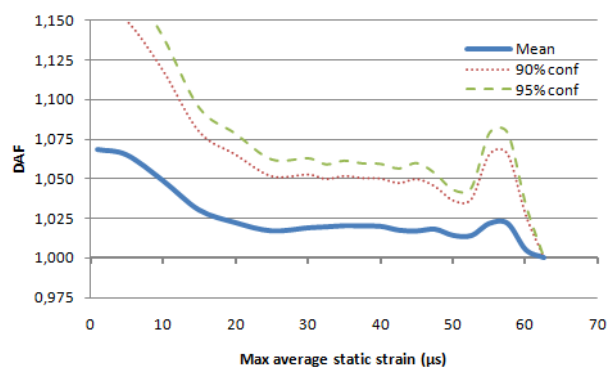
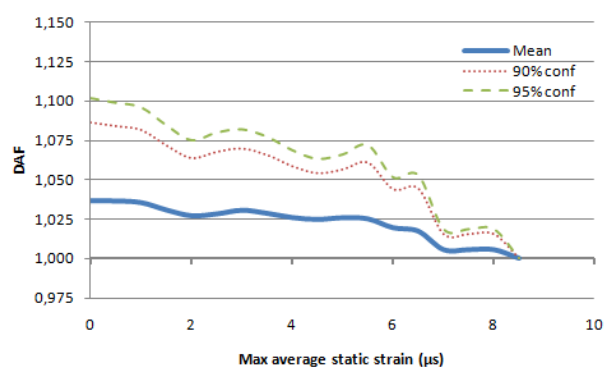


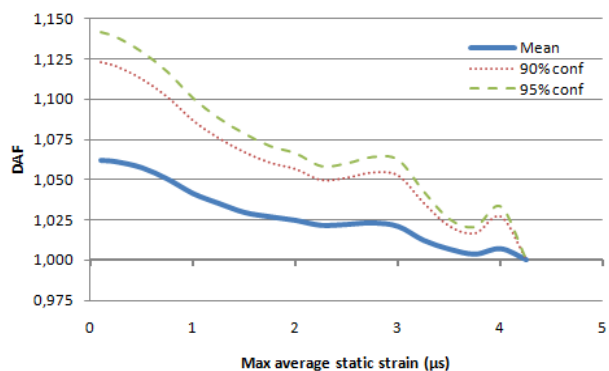
Figure 28. Measured DAF results from Vransko bridge



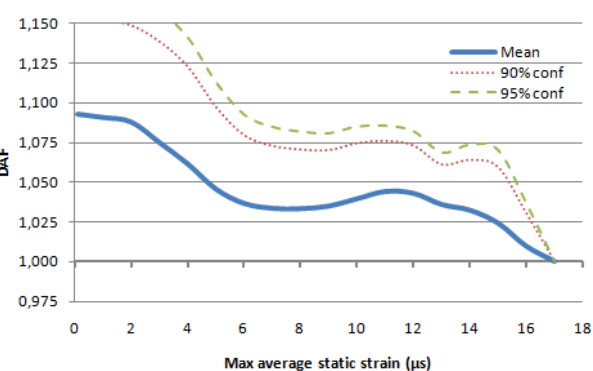
Vransko – 24.8 m simply supported span



Blagovica – 11.9 m integral slab



Trebnje – 11.5 m simply integral slab



the Netherlands – 7.0 m integral slab

Figure 29. Average values of DAF on 4 of 5 measured sites

Further investigations on dynamic allowance

Chapter 5 of the deliverable reviews a number of special topics concerning dynamic allowance that the engineer should be aware of, since it must lead to a smaller/larger dynamic allowance than anticipated a priori. These topics include the presence of a bump, the existence of pre-existing vibrations, the worst possible load effect when considering all possible bridge sections and differences between dynamic amplification factors due to normal traffic loading (i.e., 5-axle trucks) or exceptional traffic loading (i.e., cranes). The results from these theoretical investigations confirm the low dynamics associated to critical loading events already found in Chapters 3 and 4.

Influence of pre-existing vibrations

To assess the influence of bridge vibratory condition prior to heavy loading a number of alternative theoretical road surface profiles were considered. Figure 30 shows the DAF resulting from a 60-tonne single 5-axle articulated vehicle running over a 25 m simply supported bridge and compared to the same vehicle preceded by a 30-tonne vehicle that leaves the bridge in free-vibration. In the figure, the 99% DAF are plotted for a sample of 20 alternative road profiles, each with an IRI of between 1 m/km and 6 m/km. Also shown to aid visualisation are approximate upper bounds for the range of profiles considered. As can be seen the presence of pre-existing bridge vibrations increases the maximum occurring DAF for all profiles considered. The damping of the bridge in the figure is 3% and it plays an important role in the rate at which the pre-existing vibrations decay. Further details can be found in Section 5.3.

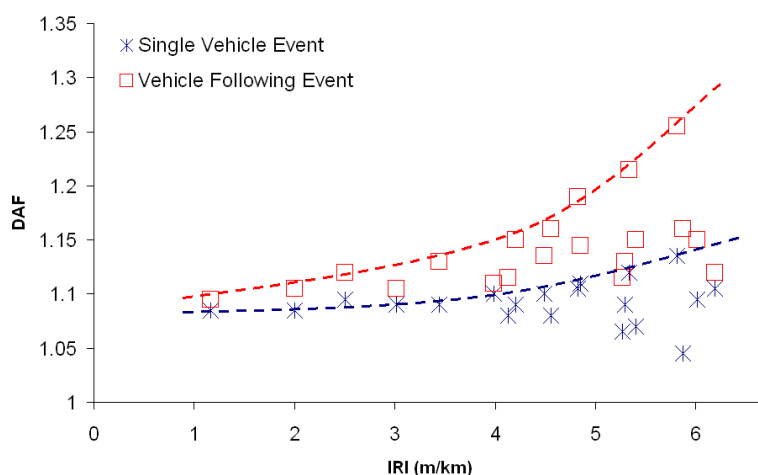


Figure 30. Influence of Pre-existing Vibrations on Dynamic Amplification

Differences in total load effect for the critical section and the midspan section

Most current research on dynamic effects due to traffic load on simply supported bridges focuses on the mid-span section of the bridge, since this location corresponds to the worst static bending moment. However, the maximum total moment may be located relatively far apart from the mid-span location and differ considerably from the maximum mid-span

moment. Section 5.4 uses Monte-Carlo simulation of the parameters of a 5-axle vehicle model travelling over an Euler-Bernoulli beam to analyse this phenomenon. DAF is defined as the maximum total bending moment at midspan divided by the maximum static bending moment at midspan. FDAF is defined here as the maximum total bending moment across the full bridge length divided by the maximum static bending moment at midspan due to the passage of a vehicle. The influence of road profile roughness and bridge length on the magnitude of the differences between mid-span and the worst possible section are also investigated and summarised in

Table 16. The results of simulations were produced with a typical European 5-axle truck configuration with a range of typical speeds and GVW based on WIM data collected in a heavily trafficked route in Auxerre (France). 100000 static events were generated and the worst 500 static events at midspan for each span were studied dynamically. Each event was studied for 100 profiles of each road profile class. The total number of calculations were 500 events x 5 bridge spans x 100 different profiles x 3 profile classes = 750000, and mean values and 95% confidence intervals are presented in the table.

Table 16 – Comparison of DAF and FDAF for different bridge lengths and ISO road classes

Bridge (m)	Road class 'A'				Road class 'B'				Road class 'C'			
	Mean		95% confid.		Mean		95% confid.		Mean		95% confid.	
	DAF	FDAF	DAF	FDAF	DAF	FDAF	DAF	FDAF	DAF	FDAF	DAF	FDAF
15	1.020	1.041	1.063	1.084	1.032	1.060	1.107	1.143	1.069	1.110	1.209	1.248
20	1.021	1.045	1.073	1.098	1.034	1.065	1.110	1.146	1.072	1.116	1.208	1.258
25	1.023	1.043	1.070	1.092	1.040	1.068	1.116	1.143	1.081	1.130	1.215	1.267
30	1.029	1.045	1.075	1.090	1.046	1.070	1.118	1.140	1.085	1.137	1.213	1.272
35	1.032	1.048	1.079	1.095	1.048	1.071	1.122	1.142	1.089	1.135	1.215	1.269

The results above are for a typical European 5-axle truck on a one-dimensional bridge beam model. Further on, Section 5.6 analyses and compares the dynamics of exceptionally heavy vehicles such as cranes to typical 5-axle European trucks using 3-D finite element models and a comparison between DAF and FDAD is provided. It is shown that when considering heavier trucks the differences between midspan and the section holding the largest bending moment will tend to be of a smaller magnitude (it was found from theoretical simulations that this difference would decrease the higher the static load effect), about a 5% dynamic increment when considering all sections of a bridge with respect to the midspan location.

Influence of a bump or expansion joint prior to the bridge

There are numerous studies on the DAFs caused by traffic flow on a bridge. For short- and medium-span bridges, the road profile appears as a dominant parameter on the bridge dynamic response. In theoretical investigations, the road profile is usually modelled as a stochastic random process. However, this approach does not take into account the high irregularities that are prone to develop in the connection of the bridge to its approach, as result of a damaged expansion joint and/or differential settlement. Section 5.5 uses planar VBI models to assess the increase in midspan moment and shear effects at the supports that a bump prior to a simply supported bridge may cause. Results for a range of bumps, bridge lengths, traffic configurations and road conditions are discussed. Two types of vehicles are analysed: a 5-axle truck and a 9-axle crane truck. The results are summarised in the three tables that follow for the case of no prior damage, a 2 cm deep and a 4 cm deep expansion joint prior to the bridge. An extended version of the tables (with more span lengths and also

the analysis of the DAF associated to shear) can be found in Section 5.5.

Table 17 -DAF for bending moment at midspan versus span length (no bump prior to the bridge)

Bridge length (m)	5-axle truck				Crane			
	Mean	S.Dev.	90% confid.	95% confid.	Mean	S.Dev.	90% confid.	95% confid.
5	1.086	0.054	1.149	1.176	1.118	0.091	1.265	1.288
10	1.057	0.041	1.109	1.128	1.028	0.040	1.085	1.104
15	1.018	0.021	1.043	1.055	1.037	0.031	1.078	1.088
20	1.048	0.028	1.081	1.092	1.030	0.028	1.066	1.081
25	1.040	0.027	1.077	1.087	1.034	0.028	1.072	1.082
30	1.042	0.024	1.077	1.086	1.035	0.029	1.069	1.085
35	1.041	0.023	1.074	1.079	1.038	0.035	1.086	1.104
40	1.037	0.020	1.061	1.074	1.042	0.039	1.094	1.118

Table 18 –DAF for bending moment at midspan versus span length (2 cms bump prior to the bridge)

Bridge length (m)	5-axle truck				Crane			
	Mean	S.Dev.	90% confid.	95% confid.	Mean	S.Dev.	90% confid.	95% confid.
5	1.271	0.170	1.521	1.625	1.134	0.090	1.277	1.295
10	1.068	0.057	1.154	1.185	1.031	0.042	1.088	1.111
15	1.021	0.023	1.046	1.054	1.040	0.031	1.081	1.099
20	1.047	0.028	1.080	1.103	1.031	0.030	1.071	1.085
25	1.045	0.027	1.081	1.089	1.034	0.028	1.070	1.081
30	1.046	0.025	1.082	1.093	1.035	0.028	1.072	1.089
35	1.046	0.027	1.084	1.094	1.036	0.030	1.075	1.089
40	1.042	0.021	1.073	1.083	1.044	0.039	1.098	1.125

Table 19 – DAF for bending moment at midspan versus span length (4 cms bump prior to the bridge)

Bridge length (m)	5-axle truck				Crane			
	Mean	S.Dev.	90% confid.	95% confid.	Mean	S.Dev.	90% confid.	95% confid.
5	1.469	0.323	1.969	2.139	1.144	0.090	1.281	1.304
10	1.123	0.092	1.254	1.280	1.032	0.043	1.087	1.112
15	1.022	0.026	1.056	1.066	1.043	0.032	1.084	1.101
20	1.042	0.028	1.072	1.087	1.037	0.031	1.076	1.088
25	1.042	0.025	1.074	1.089	1.036	0.031	1.080	1.091
30	1.054	0.027	1.092	1.102	1.031	0.028	1.061	1.075
35	1.050	0.025	1.084	1.092	1.037	0.035	1.077	1.109
40	1.044	0.023	1.075	1.085	1.047	0.042	1.104	1.130

When there is no damaged expansion joint, the cranes may exhibit a slightly higher dynamic component than 5-axle trucks for some span lengths (Table 17). The presence of a damaged expansion joint increases the overall DAF for both type of vehicle configurations, but the 5-axle truck is far more sensitive than the crane, particularly for shorter spans (Table 18 and Table 19).

Dynamic allowance for exceptionally loaded vehicles (cranes)

VBI is often considered for the most common classes of vehicle such as the 5-axle articulated truck. However, the dynamic response of bridges to this type of trucks is quite different from the bridge response to the vehicles more likely to feature in maximum-in-lifetime traffic loading events. Section 5.6 focuses on large (>100 tonne) cranes and crane-type vehicles that have been recorded at WIM sites in Europe (Deliverable D08). Here, the total bending moment due to these vehicles on short to medium span bridges is compared to 5-axle articulated trucks using 3-D VBI FE models. To account for the variability in vehicle characteristics, more than 40000 VBI events are computed using Monte Carlo simulation based on 77 vehicles (77 worst 5-axle trucks and 77 worst cranes) generating the daily maxima loading effect. Four spans are considered, this is, 7.5, 15, 25 and 35 m. For the 7.5 m bridge, two boundary conditions were analysed: fixed-fixed and simply supported. Vehicle and bridge are represented using 3-D FEM. Three ISO class 'A' road profiles were considered within the simulations (there were no significant differences among the results of the 3 profiles). Variability was allowed in vehicle mechanical properties, speeds and mass distribution. Table 20 summarises the results. It must be noted that when assessing a bridge close to a fixed-fixed support condition (i.e., an integral type), the DAF values will be significant lower than a simply supported condition or those general recommendations given in bridge codes.

Table 20 - DAF for bending moment versus span length (m)

Bridge (m)	Boundary condition - Section	5-axle truck				Crane			
		Mean	Stand. Dev.	90% confid	95% confid	Mean	Stand. Dev.	90% confid	95% confid
7.5	Fixed-Fixed - support	1.004	0.032	1.038	1.047	1.000	0.025	1.030	1.039
7.5	Fixed-Fixed - midspan	1.000	0.037	1.041	1.053	0.999	0.031	1.036	1.047
7.5	Simply Supp. - midspan	1.008	0.039	1.051	1.063	1.022	0.027	1.055	1.066
15	Simply Supp. - midspan	1.015	0.037	1.067	1.087	1.015	0.017	1.036	1.043
25	Simply Supp. - midspan	1.022	0.044	1.079	1.101	1.014	0.025	1.044	1.056

It has been observed the scatter of the DAF distribution generally increases for longer bridge spans. When comparing both types of vehicles, the most frequent DAF values for cranes are smaller than for 5-axle trucks. Therefore, the histograms of DAF versus number of occurrences

are narrower for cranes than for 5-axle trucks.

Table 21 shows the equivalent of Table 20 when considering the worst possible section, this is, FDAF. The results show similar standard deviation for DAF and FDAF. Generally, the higher the static loads the smaller the difference between DAF and FDAF becomes. In this table, all FDAF values for cranes remain below 1.1.

Table 21. DAF for midspan bending moment versus span length (m)

Bridge (m)	Boundary condition - Section	5-axle truck				Crane			
		Mean	Stand. Dev.	90% confid .	95% confid .	Mean	Stand. Dev.	90% confid .	95% confid .
7.5	Fixed-Fixed	1.007	0.039	1.048	1.06	1.009	0.032	1.046	1.058
7.5	Simply Sup.	1.021	0.040	1.066	1.079	1.028	0.027	1.061	1.072
15	Simply Sup.	1.025	0.036	1.076	1.095	1.016	0.017	1.037	1.045
25	Simply Sup.	1.058	0.043	1.119	1.138	1.022	0.024	1.052	1.063

Conclusions and final recommendation

Site specific assessment of traffic loading has considerable potential to prove that bridges are safe which would otherwise have been rehabilitated or replaced. This is due to the conservatism of bridge standards that cover a wide range of possible traffic loading conditions throughout the road network. Deliverable D10 has shown how dynamic allowance for traffic loading on an existing bridge can be determined using validated VBI FE models (Chapter 3), but they require bridge drawings, measured bridge properties and road profile, and updated WIM data for the site. Alternatively, the dynamic allowance can be experimentally derived from measured total load effects and the associated dynamic component using modern Bridge-WIM technology (Chapter 4). A simpler approach is to adopt the large dynamic allowance given in bridge codes (Chapter 2) that must cover for the many variables and uncertainties associated to the VBI problem.

There is clearly a considerable gap between the complex mathematical modelling and experiments required for an accurate determination of dynamic allowance and the conservative values available at bridge codes. In order to reduce this gap, ARCHES proposes an intermediate solution based on the large amount of experimental tests and numerical simulations carried out during the project. The quality of the road profile plays a role that becomes more dominant as the span length decreases, but in the case of very good road profiles (ISO class 'A'), the critical loading cases governing the maximum load effects typically produce dynamic amplification factors below 1.1. Nevertheless, the presence of a bump or a damaged expansion joint prior to the bridge may lead to higher values in short span bridges (Section 5.5). Even so, it has been shown that exceptionally heavy vehicles representing critical loading cases such as cranes, have a rigid configuration that generates smaller dynamics than typical 5-axle articulated trucks (Section 5.6). So, if the road profile of a bridge was maintained in a good condition, the dynamic amplification factor associated to the critical loading cases could be substantially reduced in relation to the values built within the Eurocode traffic load models.

General recommendations are provided for assessment of 1-lane and 2-lane bridges (both moment and shear load effects) with ISO road classes 'A' and 'B' (Figure 31). The values provided in the recommendations represent an upper envelope that covers for a large amount of Monte-Carlo simulations varying road profile and traffic static and dynamic properties. These recommendations take into account the maximum total load effect for the entire bridge

length (or FDAF as described in Section 5.4). For assessment of 1-lane and 2-lane bridges with a road class ‘A’ (both moment and shear load effects), ARCHES recommends DAF values that varies linearly from 1.3 for a 5 m bridge to 1.15 for a 15 m bridge. Then, DAF remains constant at 1.15. For road class ‘B’, the DAF recommendation also varies linearly between 1.4 and 1.2 from 5 m to 15 m respectively. Then, it remains constant at 1.2. The recommended dynamic allowance represents a significant reduction with respect to the 1-lane values built within the Eurocode traffic load models for both road classes. For 2-lane bridges, the recommended values are also smaller than Eurocode values if the road profile was a class ‘A’.

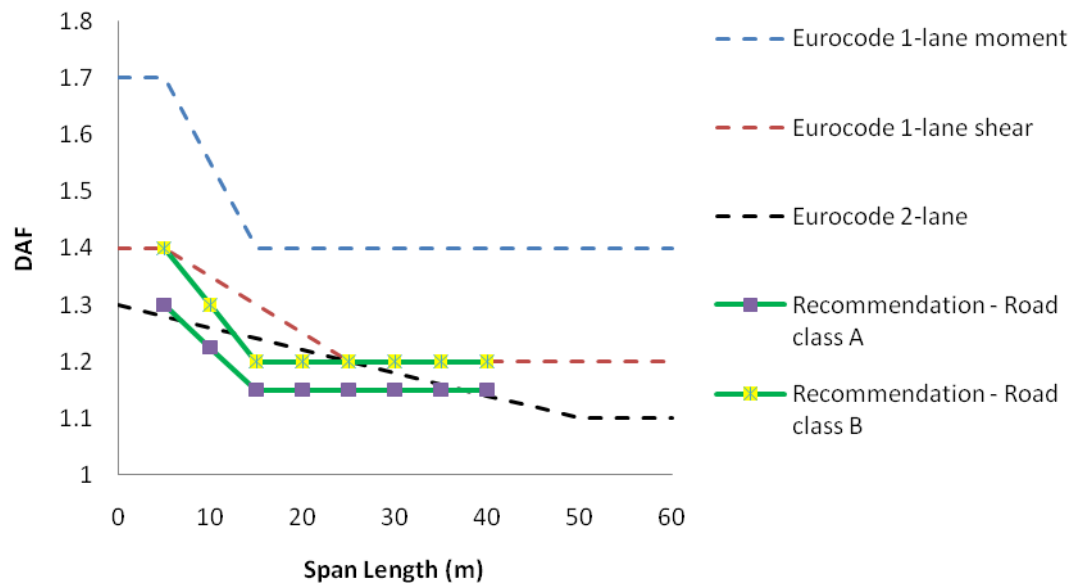


Figure 31. DAF recommendation versus Eurocode values

Finally, further reductions in dynamic allowance can be achieved if a better knowledge of the bridge response was acquired through numerical simulations and field tests (Chapters 3 and 4). Most probably, measurements will show that DAF is considerably less than what is reflected in Figure 31. In fact, the five bridge measurements on heavily trafficked motorways carried out within the ARCHES project, lasting from 2 weeks to 2 months, on three integral slab bridges, on one simply-supported beam-deck bridge and on one 6-span simply-supported beam bridge with a continuous deck, consistently showed that the DAF decreased as a function of increasing weight of the loading events. The average DAF values for the extremely heavy vehicles (low-loaders or cranes) and for the multiple presence events with 2 heavy vehicles, was on all 5 measured sites close to 1. The DAF of the heaviest loading events in 3 out of 5 sites was equal to 1.0, and only on a longer simply supported beam-deck bridge, where the heaviest event was caused by 2 heavy semi-trailers simultaneously on the bridge, the DAF reached 1.04. On that site a considerable bump was measured on the approach to the bridge, which excited VBI.

If performed, DAF measurements will optimise assessment of the existing bridges, because:

- the measured DAF values will be likely much lower than those prescribed in the design codes and consequently,
- because knowing the real DAF reduces uncertainties of the structural safety assessment which can be employed through lower safety factors for traffic loading.

However, the proposed envelope of dynamic effects of Figure 31 offers an inexpensive way to

give a preliminary realistic assessment of the dynamics of a bridge purely based on its length and the road class.

3.1.7 Systematic decision making processes associated with maintenance and reconstruction of bridges

Introduction

The Deliverable D09 - Recommendations on systematic decision making processes associated with maintenance and reconstruction of bridges - was prepared in the work package 2: Structural Assessment and Monitoring.

The main goal of the report is to collect information about decision making processes associated with maintenance and reconstruction of bridges and to prepare a recommendation.

The recommendation was prepared on the base of collected information and it is concentrated on New Member States (NMS) and Central and Eastern European Countries (CEEC).

The report is divided into five main parts:

- description of basic structure of bridge management system - BMS (chapter 2),
- literature review of research reports and conference papers (chapter 3.1 and 3.2),
- national reports (chapter 3.3),
- questionnaire survey (chapter 3.4),
- recommendation (chapter 4).

Different modules used within ordinary bridge management system are described in chapter 2. It includes the following modules: administration, inventory, inspection, maintenance and prioritisation. The basic function of each module is introduced there together with usage of catalogue of defects and cost catalogue. The main contribution of Bridge life cycle cost analysis is presented together with basic cost categories: agency, user and other costs. The differences in decisions made on bridge (project) level and network level are described.

The summary of selected research reports issued from 2000 and concentrated on bridge assessment and bridge management is mentioned in chapter 3.1. It includes the deliverables of FP4 project BRIME, deliverables of FP5 projects SAMARIS and SAMCO, COST 345 report, UK code of practice and two American NCHRP reports.

In chapter 3.2 are presented selected conference papers from 2008. It include mainly description of bridge management strategy in Hungary, Croatia, Portugal and Austria (3rd European Pavement and Asset Management Conference), LCCA algorithm for bridges (87th annual meeting of Transportation Research Board) and safety of existing bridges (Transport Research Arena).

National reports describing the situation concerning bridge management system and related decision making processes are presented in appendix A; the basic information is summarized in chapter 3.3. It includes characterization of BMS in Bulgaria, Czech Republic, Estonia, France, Italy, Latvia and Slovakia.

Results of the questionnaire survey are summarized in chapter 3.4. The basic information from Germany, Serbia, Ukraine and UK are introduced there to supplement information about already mentioned countries. The empty questionnaire form is presented in appendix B and the answers from different countries are presented in appendix C.

The results are summarized in recommendation in chapter 4. It includes four topics: how to connect the current system to new BMS, what is the recommended structure of BMS, what are the main decision making processes within BMS and how to connect the BMS to decision systems of other assets.

3.1.8 Validation and application of low-alloy steel - Recommendations for the use of corrosion resistant reinforcement

Introduction

The objective of Recommendations is to provide methods and techniques that will improve the durability of new and existing concrete structures in NMS. In order to make these methods applicable in civil engineering practice, one of the key issues is their economic justification. Nowadays, there are numerous methods and techniques for the repair and rehabilitation of degraded reinforced concrete structures. They can generally be divided in the following two groups: (1) which focuses on improving concrete and (2) which focuses on improving reinforcement properties. One of the possibilities for the improvement of reinforcement performance in concrete is the use of certain types of corrosion resistant steel as selective or total substitution of black steel reinforcement. These possibilities of utilization of corrosion resistant reinforcement in concrete were explored and evaluated within Workpackage 3 of the ARCHES project.

The first step consisted in an investigation of a series of corrosion resistant steel types available on the market which were then classified with respect to their content of alloying elements. The aim was to identify those types of steel that contained lower content of alloying elements, and would consequently be lower in price, but still have satisfying corrosion behaviour. When potential steel types were chosen and obtained, a detailed experimental program was created, which consisted of thorough research of corrosion behaviour of all steel types as follows: (I) in solutions simulating concrete, (II) in concrete at laboratory controlled aggressive conditions and (III) in concrete in real aggressive environment.

The working task was divided into the following subtasks:

State of the art research

- literature review
- definition of steel types which bear the most potential for research
- reinforcing steel producer survey, supply of different types of corrosion resistant steel

Laboratory testing in pore solution

- corrosion testing of different types of steel (small electrodes)

Laboratory testing in concrete

- accelerated corrosion testing of steel in small concrete samples
- corrosion testing of steel in mid-size concrete beams

Establishment of exposure site

- selection of test location
- preparation of samples for long-term on-site trials
- establishment of exposure site

During the course of the project, corrosion resistance of chosen types of steel was compared to black steel known to have very weak **Błąd! Nie można odnaleźć źródła odwołania.** corrosion resistance, and with highly alloyed types of stainless steels, the corrosion resistance of which is known from previous laboratory research **Błąd! Nie można odnaleźć źródła odwołania.****Błąd! Nie można odnaleźć źródła odwołania..**

The guidelines have been developed on the basis of the results that were obtained during the three years of the ARCHES project's duration, and on a detailed literature survey of practical experience in using corrosion resistant steel reinforcement throughout the world.

Summary of test results

Results that were achieved and evaluated during the ARCHES program include a laboratory study of rebars in pore solution, rebar in concrete examination and two field tests of columns and a large concrete reinforced structure.

Two different techniques were used for laboratory tests in synthetic pore solutions of the test

materials. The potentiodynamic technique was applied in the tests of fresh polished cross sections, whereas the tests of corrosion behaviour of external surfaces in synthetic pore solution were studied using electrochemical impedance spectroscopy. Since the test parameters were different (the state of the passive film and the two different techniques), the conclusions drawn from measurements may be compared, the trend may be observed, but results have to be interpreted relatively.

The passive film on the cross section was first removed from the surface and it was formed during 1h at open circuit potential prior to anodic polarization. At high values of pH (12) all materials performed well as no significant differences were detected. In the presence of even small amounts of chloride ions (0.3 %), the performance of TOP12 was weakened. In the presence of high amounts of chlorides (1 %), however, the following (corrosion performance) trend was observed (

Table 22):

Black steel (not passive) $\leq$ TOP 12 $\leq$ 204 Cu, AISI 304 and AISI 304L $\leq$ SAE/UNS S3 2205 and UGIGRIP 4362.

Table 22 Qualitative estimation deduced from anodic polarization measurements for the different tested steels in pore solution pH 12.4 and pH 10.1 (Table 25 and Table 26 in Appendix A)

Total impedance, $k\Omega\text{ cm}^2$	pH 12.5			pH 10.0		
Steel type / Cl^- content	0%	0.3%	1%	0%	0.5%	1%
Black steel	√√	√	×	×	×	×
TOP12	√√√	√√	√	√√	√	×
204Cu	√√√	√√√	√√	√√√	√√	√
AISI 304	√√√	√√√	√√	√√√	√√	√
AISI 304L	√√√	√√√	√√	√√√	√√	√
SAE/UNS S3 2205	√√√	√√√	√√√	√√√	√√√	√√√
UGIGRIP 4362	√√√	√√√	√√√	√√√	√√√	√√

× - not acceptable performance

√ - acceptable performance

√√ - good performance

√√√ - excellent performance

At pH 10 the conditions resemble a carbonated environment, the black steel did not show any passive behaviour, TOP12 (1.4003) is strongly affected by the decrease of alkalinity. The rest of the stainless steels performed well. Corrosive behaviour of 204Cu is comparable to austenitic and duplex stainless steels. At pH 10 and the small amount of chlorides present (0.5 %), TOP12 (1.4003) can not be used, whereas 204Cu behaves similarly to austenitic stainless steels. Both duplex stainless steels showed optimal resistive behaviour:

Black steel and TOP 12 (not passive) $\leq$ 204 Cu, AISI 304 and AISI 304L $\leq$ SAE/UNS S3 2205 and UGIGRIP 4362 (1.4362).

For external surface samples, where the passive film is in its own form as provided from the manufacturers, Black steel and TOP12 (1.4003) did not show good performance neither at high (12) nor low (10) pH. 204Cu is acceptable in very alkaline and carbonated environments both. AISI 304 (1.4301), SAE/UNS S3 2205 (1.4462) and UGIGRIP 4362 (1.4362) show

similar behaviour. Among the tested materials, AISI 304L (1.4306) exhibits the highest resistivity towards corrosion process at pH 12.5 and 10.0 with or without chlorides.

At pH 12 and the presence of chlorides (1 %) the performance can be described as follows:

Black steel and TOP 12 (not passive) $\leq$ 204 Cu $\leq$ AISI 304, SAE/UNS S3 2205 and UGIGRIP 4362 $\leq$ AISI 304L.

Results are also presented in the

Table 23.

At pH 10 and the presence of chlorides the performance can be described as follows (

Table 23):

Black steel and TOP 12 (not passive) $\leq$ 204 Cu $\leq$ AISI 304 and UGIGRIP 4362 $\leq$ SAE/UNS S3 2205 and AISI 304L

It is to be noted that 204Cu performed even better at lower pH (10) compared to higher pH (12.5) of pore solution, as also observed in the reference literature.

Table 23 Qualitative estimation deduced from electrochemical impedance spectroscopy results for all tested steel specimen at two different pH of pore solution with different additions of chloride ions (Table 27 in Appendix A).

Total impedance, $k\Omega\text{ cm}^2$	pH 12.5			pH 10.0		
Steel type / Cl ⁻ content	0%	0.5%	1%	0%	0.5%	1%
Black steel	√	×	×	×	×	×
TOP12	√	√	√	×	×	×
204Cu	√√	√√	√√	√√	√√	√√
AISI 304	√√	√√√	√√√	√√	√√	√√
AISI 304L	√√	√√√	√√√	√√√	√√√	√√√
SAE/UNS S3 2205	√√	√√	√√	√√	√√√	√√√
UGIGRIP 4362	√√	√√	√√	√√	√√	√√

× - not acceptable performance

√ - acceptable performance: 1-100 $k\Omega\text{ cm}^2$

√√ - good performance: 101-800 $k\Omega\text{ cm}^2$

√√√ - excellent performance: $\geq 801\text{ k}\Omega\text{ cm}^2$

The presented results indicate that electrochemical properties of external surfaces of tested steels are different from those obtained in the cross section sample experiment. If the passive films of all different types of rebars are in excellent conditions, the results would be similar to those, obtained on cross sections with potentiodynamic curves (Table1). The films that form during 1 h of exposure to environment after the samples are abraded, represent a natural way of film formation. The passive films can be, thus, qualitatively compared. On the other hand, results from external exposure of specimen to alkaline environment, showed somehow different behaviour. The difference in electrochemical behaviour of TOP12 and UGIGRIP 4362 sample, when abraded or with a passive film is caused by an improper treatment of steel rebars. A visual examination of rebars proved that TOP12 (1.4003) and UGIGRIP 4362 (1.4362) suffered from poor handling conditions.

Corrosion behaviour of steel rebars in concrete was examined in three different ways: by exposure of concrete specimens to a salt spray chamber, by laboratory tests of concrete specimens and by using embedded probes and rebars in concrete specimens. Corrosion

potential measurements and the galvapulse technique were used to evaluate the corrosion state in specimens during their 8 months of exposure.

Concrete specimens exposed to wetting/drying and salt spray cycles show the lowest resistance to chloride induced corrosion in dry/humid environment in black steel and TOP12 (1.4003) grade ferritic stainless steel. A significant decrease in corrosion potential and an increase in corrosion current was observed after 4 and 8 months of exposure for black steel and TOP12 (1.4003), respectively. Other tested types of stainless steels maintain a stable state during the time observed, as is presented below:

Black steel $\leq$ TOP 12 $\leq$ 204 Cu, AISI 304 and AISI 304L, UGIGRIP 4362 and SAE/UNS S3 2205.

Table 24 Concrete specimens in salt spray chamber: mean values of corrosion potentials of the steel rebars vs. time (Ag/AgCl2 half-cell)(Table 30 in Appendix A)

Time [months]	4	6	8
Steel grade	E_{corr} [mV]	E_{corr} [mV]	E_{corr} [mV]
Black steel	-245	-291	-409
TOP12	-132	-183	-319
204Cu	-77	-107	-94
AISI 304	-126	-123	-122
AISI 304L	-123	-105	-130
SAE/UNS S3 2205	-83	-107	-102
UGIGRIP 4362	-91	-128	-108

The above described laboratory exposure of concrete specimens reinforced with steel rebars and embedded ER probes to chloride environment show that the time required for the initiation of a corrosion process on samples with black steel and TOP12 (1.4003) is relatively short. A similar trend can be observed in specimens with a transversal crack. The 204 Cu low nickel stainless steel has a low corrosion potential which remained stable. Other test materials (AISI 304 – 1.4301 and AISI 304L – 1.4306 and UGIGRIP 4362 – 1.4362 and SAE/UNS S3 2205 – 1.4462) have better corrosion resistance to chloride attack in a concrete environment. Their potentials remain stable and their corrosion rates low during the observed time of exposure according to the trend described bellow:

Black steel and TOP 12 $\leq$ 204 Cu, AISI 304 and AISI 304L, UGIGRIP 4362 and SAE/UNS S3 2205.

Table 25 Concrete specimens with embedded ER probes rebars: mean values of corrosion potentials of the steel rebars vs. time (Cu-CuSO4 half-cell).

Time [months]	4	6	8
Steel grade	E_{corr} [mV]	E_{corr} [mV]	E_{corr} [mV]
B500B	-329	-417	-444

TOP12	-205	-342	-397
AISI 304	-202	-229	-222
AISI 304L	-201	-323	-222
204Cu	-229	-263	-364
SAE/UNS S3 2205	-197	-214	-209
UGIGRIP 4362	-225	-264	-259

Table 26 Concrete specimens with artificial transverse cracks: corrosion potentials of the steel rebars vs. time (Cu-CuSO₄ half-cell).

Time [months]	4	6	8
Steel grade	E_{corr} [mV]	E_{corr} [mV]	E_{corr} [mV]
B500B	-639	-662	-774
TOP12	-501	-528	-536
AISI 304	-234	-224	-206
AISI 304L	-205	-367	-262
204Cu	-403	-464	-464
SAE/UNS S3 2205	-186	-216	-229
UGIGRIP 4362	-228	-242	-234

The scope of introducing large concrete specimens with steel reinforcement into the research was to confirm the laboratory test results in harsh marine environments as well.

The large specimen were cyclically exposed to the of sodium chloride solution environment. The chloride concentration at the rebar or bars level was around 0.1% by weight of concrete and exceeds the value corresponding to the initiation of corrosion for the carbon steel..

The three alloys were tested. The two stainless steels, AISI 304 (1.4301) and AISI 2205, behave very well, showing almost negligible corrosion rate values during the whole testing period in comparison to common black steel. TOP12 (1.4003) alloy showed corrosion rates approximately half of those of the carbon steel.

The time of exposure of large concrete columns (at the Krk bridge) was insufficient to present any relevant conclusions at the time. Only initial measurements were conducted and no initiation of corrosion process had taken place by the time the project ended. Further investigation and monitoring are being performed.

3.1.9 Development and application of cathodic protection system - Guideline for Smart Cathodic Protection of steel in concrete.

Introduction

Unplanned maintenance of ageing civil engineering structures and in particular concrete bridges is a major problem, among others for organisations responsible for road networks in

Europe. Especially corrosion of reinforcement is widespread, which raises questions about structural safety. Repair of corrosion related damage is very costly and may cause a structure to become unavailable for prolonged periods of time.

As part of the ARCHES project, this report addresses Cathodic Protection (CP) as an alternative approach to remediation of corrosion with considerable benefits. Benefits may be lower cost over the whole life, shorter execution time, longer working life of the intervention and increased durability and safety.

Owners of large stock of bridges and/or their consulting engineers need to know the basics of corrosion of reinforcement, concrete repair methods and CP. Further elaboration of repair and CP are specialised activities that may be left to technical specialists. On a general level, owners should be aware of the long and successful track record of CP of concrete and its flexibility to suit the needs of individual structures.

This report first sketches the technical background and provides examples, focusing on bridges with corrosion damage and the solutions provided. Then a Technical Guideline is presented with the major steps described that have to be made to apply CP. Next, CP is placed in a wider framework of concrete maintenance, including economic considerations and case studies of whole-life costs. The Annex provides descriptions of trial CP systems that were investigated and some typical examples of bridges that suffer corrosion.

The conclusions to this study can be summarised as follows.

Cathodic protection (CP) of reinforcing steel has been applied to concrete structures with corrosion damage for over 25 years. World wide experience shows that CP prevents further development of corrosion damage in a reliable and economical way for a long time, provided that the CP system is designed, executed and maintained properly.

Since the 1980's, CP has been applied to buildings, marine structures, tunnels and bridge decks and substructures in the US, Europe, the Middle East, Asia and elsewhere in the world. New anode materials have become available, of which in particular activated titanium and conductive coatings have proved their good performance over more than two decades. CP of concrete structures has been standardised in the US since the 1980s and in Europe since the year 2000.

As a wide variety of anode systems is available with a good track record and tailor made solutions can be provided for every type of structure.

The principle of CP is active and permanent intervention in the corrosion process. Therefore, monitoring is an essential part of operating a CP system. The main advantage is that monitoring proves the absence of corrosion on a regular basis.

Designing a CP system for a particular structure requires that proper information is available on the structure, in particular about the extent of damage and the layout of the reinforcement. CP can be applied to both reinforced and prestressed concrete structures. Application to post-tensioned structures can be done on a routine basis. It is recommended to perform a trial in case of slender (low cover) pre-tensioned elements. In all cases, very negative potentials of stressed high strength steel should be avoided, as they may cause loss of ductility. Dedicated monitoring of pre-tensioned steel or post-tensioning ducts in prestressed structures is needed to avoid overprotection. Numerical modelling of CP has been demonstrated to be a useful tool, both for predicting CP operation in general and the safety of potentials of prestressing steel in particular.

CP systems have a long working life. As growing experience has shown, typical life of conductive coatings systems is more than 10 years. The life of typical activated titanium systems is at least 25 years and probably much more. Corrosion of reinforcement will be completely absent during this period and probably many more years after the end of the working life. The long life of CP is in strong contrast to the life of conventional repairs. European studies and typical examples have shown that conventional repairs may have short

lives of as low as five to ten years, among others due to poor execution quality. The working life of repairs is highly uncertain and the absence of corrosion and potentially severe loss of cross section cannot be guaranteed.

CP fits into a rational maintenance strategy in various ways. Both from technical and economical points of view, the recommended scenario is that corrosion and concrete damage are detected in a relatively early stage and that subsequently CP is applied. Waiting until damage has become extensive is unfavourable, as it will increase total costs. Applying CP before damage has appeared may be favourable in individual cases only.

Based on case studies, the cost of CP over the (remaining) life of a structure has been investigated and compared to other options. Generalising, it appears that CP may be instrumental in saving considerable amounts of money over the remaining life of a structure, say over periods of 10 to 25 years.

3.1.10 The use of prestressed externally glued FRP

Introduction

The “Recommendations for pre-stressed externally glued FRP strips” mainly cover the structural strengthening of bridge components of onshore structures, using pre-stressed carbon fibre reinforced plastics (CFRP). The Recommendations give advice on the selection of laminate materials, analysis, design and implementation of strengthening. In some cases it may be advantageous to bond the external FRP reinforcement onto the concrete surface in a prestressed state. Both laboratory and analytical research shows that prestressing represents a significant contribution to the advancement of the FRP strengthening technique, and methods have been developed to prestress the FRP composites under real life conditions.

Pre-stressing the strips prior to bonding has the following advantages:

- Provides stiffer behaviour as at early stages most of the concrete is in compression and therefore contributing to the moment of resistance.
- Crack formation in the shear span is delayed and the cracks when they appear are more finely distributed and narrower (crack widths are also a matter of bond properties).
- Closes cracks in structures with pre-existing cracks.
- Improves serviceability and durability due to reduced cracking.
- Improves the shear resistance of member as the whole concrete section will resist the shear, provided that the concrete remains uncracked.
- The same strengthening is achieved with smaller areas of stressed strips compared with unstressed strips.
- With adequate anchorage, pre-stressing may increase the ultimate moment of resistance by avoiding failure modes associated with peeling-off at cracks and the ends of the strips.
- The neutral axis remains at a lower level in the pre-stressed case than in the unstressed one, resulting in greater structural efficiency.
- Pre-stressing significantly increases the applied load at which the internal steel begins to yield compared to a non-stressed member.
- The technique has also some disadvantages:
- It is more expensive than normal strip bonding due to the greater number of operations and equipment that is required.
- The operation also takes somewhat longer.
- The equipment to push the strip up to the soffit of the beam must remain in place until the adhesive has hardened sufficiently.

The concept for applying a pre-stressed FRP strip is shown schematically in Figure 32 and a schematic illustration of the stressing device is given in Figure 33.

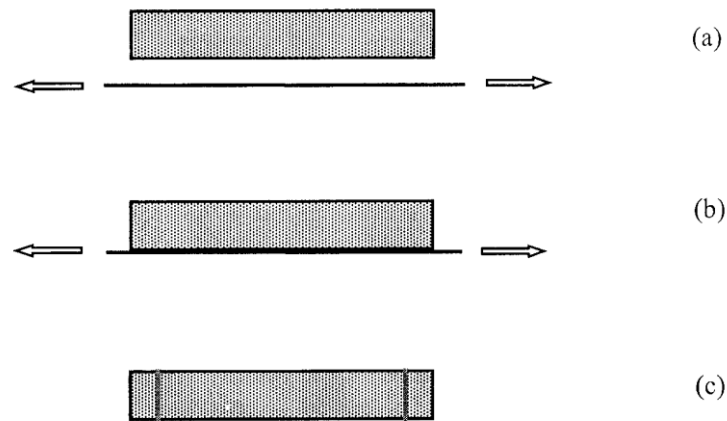


Figure 32. Strengthening with pre-stressed CFRP strips: a) prestressing, b) bonding, c) end anchorage and FRP release upon hardening of the adhesive

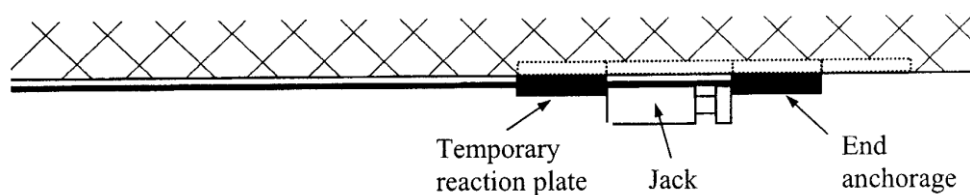


Figure 33. Schematic illustration of active anchorage.

When the prestressing force is too high, failure of the beam due to release of the prestressing force will occur at the two ends, due to the development of high shear stresses in the concrete just above the FRP. Hence the design and construction of the end zones requires special attention. Tests and analysis have shown that if no special anchorages are provided at the ends, FRP strips shear-off (from the ends) with prestress levels in the order of only 5-6% of their tensile strength (for CFRP). But a technically and economically rational prestress would require a considerably higher degree of prestressing, in the range of 50% of the FRP tensile strength, which may only be achieved through the use of special anchorages applying vertical confinement (see Figure 32 c). Such systems have been developed for practical applications as well as research purposes (

Figure 34).

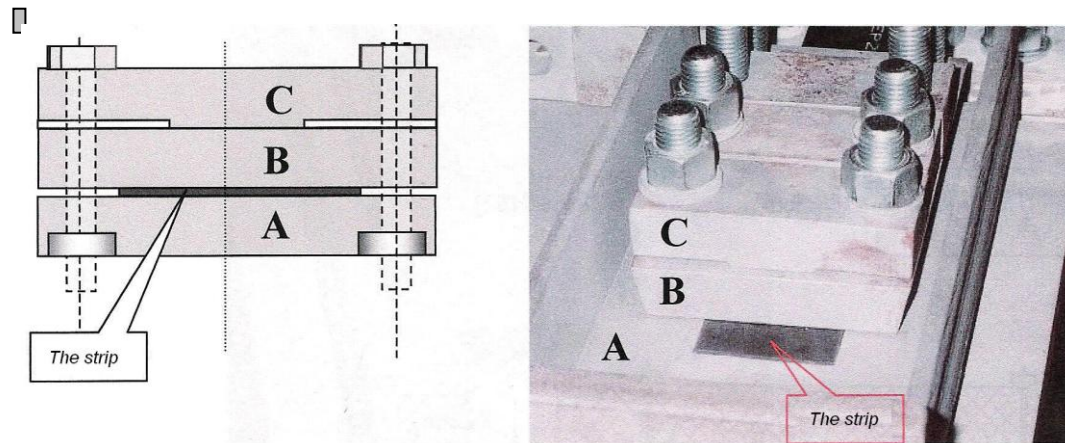


Figure 34. Special anchorages

3.1.11 Development of UHPFRC from local components - Recommendations for the tailoring of UHPFRC recipes for rehabilitation.

Introduction

The wide dissemination of Ultra High Performance Fibre Reinforced Concrete (UHPFRC) technology, specially in very demanding applications such as cast-in situ rehabilitation works requires UHPFRC formulations from local components. However, it is extremely difficult to achieve sufficient workability just by replacing cement and plasticizer from existing optimized UHPFRC recipes by locally available ones. Insufficient workability most often either forces to increase water dosage and water/binder ratio which severely decreases all performances of UHPFRC or also prevents the use of a sufficient fibrous mix to achieve tensile strain hardening. On another hand the very low water/binder ratio of UHPFRC in the range of 0.2 or less induces a very low degree of hydration of cement grains at long term (typically 0.3 to 0.5). Thus most of the cement in Ultra High Performance Concrete (UHPC) matrices is used for packing and workability but will never contribute to hydration, at best to self healing properties. Further, most cement-superplasticisers compatibility problems are related to negative interactions between cement chemical components (typically reaction products of C_3A and sulphates) and the dispersive action of superplasticisers.

It is thus of interest to investigate possibilities to replace very significant parts of the reactive cement grains in UHPC matrices by other grains, that have a more “neutral” or even positive response towards the superplasticisers and still exhibit a morphology and size distribution close to that of the cement, without “disturbing” to a significant extend the original packing. Limestone fillers are excellent candidates for this purpose.

In this perspective, UHPFRC mixes with replacement of 50 % of the cement by limestone filler has been tested and applied successfully in this study. Strain hardening UHPFRC recipes with excellent tensile and protective properties could be produced with locally available components from Slovenia on one hand and Poland on the other hand. All properties including shrinkage and mechanical response under restraint were checked and the mixes showed properties comparable or better to the original recipes with pure CEM I, developed for similar applications, during project SAMARIS. This concept opens up very promising possibilities to produce UHPFRC with locally available components without losing significantly on any property neither at fresh state nor at hardened state.

Methodology for mix design

The goal of the mix design is to achieve UHPFRC recipes with satisfactory properties for rehabilitation applications, with respect to three aspects summarized as "**PMW**":

- **Requirement "P": Protective function at serviceability:** dense matrix with very low permeability to fluids and gases, very low capillary water absorption, and no macrocracking (only finely distributed microcracks, barely visible to the naked eye can be tolerated at serviceability to guarantee the continuity of the protective function of the UHPFRC).
- **Requirement "M": Mechanical performance:** high uniaxial tensile strength (in the range of 10 MPa), and deflection or tensile strain hardening response (deformability of 0.5 to 3 ‰) according to the requirements of the application foreseen (considering orientation effects of the fibres, geometry and conditions of casting such as space available in formworks, etc.).
- **Requirement "W": Workability – rheology:** acceptable mixing time, self compacting character, if required tolerance to slopes or passing ability to fill complex or narrow formworks, 2 to 3 hours minimum range of performance (from water addition in mixer) without significant loss of workability.

Guidance for the choice:

Two major kinds of applications for rehabilitation of structures can be distinguished:

Prefabricated elements applied on the existing structure. in this case, provided the formworks do not have complex shapes with holes for instance, restraint of the shrinkage deformations at early age is not hindered and the dominating load case is bending during transport and local impact. In such a case, deflection hardening UHPFRC with "regular" fibre dosages around 2 % vol. are likely to be sufficient.

Cast-on site applications of UHPFRC overlays on existing structures. In this case, shrinkage deformations at early age are restrained to a more or less large extent by the existing structure, which gives rise to very high tensile stresses (up to 10 MPa). To guarantee crack control with finely distributed cracks even if the matrix cracking strain is reached in such a case, the UHPFRC must exhibit a tensile strain hardening response in the structural member. This requires UHPFRC mixes with low dispersion of properties and high fibre dosages up to 6 % vol. Further in those applications, the tensile strength of the materials is also a key parameter. Additions of micro fibres such as steel wool to increase the apparent tensile strength is most suited for this purpose.

The design of a UHPFRC

The methodology for the design of a UHPFRC recipe can be summarized as follows:

1. Choice of the fibrous mix: length, shape, material, aspect ratio and dosage of the fibres
2. Choice of the binder, mineral additions, ultrafines (type and dosage)
3. Choice of the superplasticizer that offers the maximum water reducing efficiency for a given workability and determination of its dosage at saturation¹.

¹ When the dosage of superplasticizer is progressively increased, everything else kept constant in the recipe, the workability increases more or less. For too low or too high dosages, the effect of a change is barely noticeable, in

4. Choice of the aggregates and paste content according to fibre dosage and workability requirements.
5. Adjustments of Water/Fines, Ultrafines/Fines, fibrous mix, and paste content to satisfy combined requirements "PMW".

Choice of fibres

Key parameters for the choice of a fibre are: **length**, **material**, **geometry** (shape, surface condition-smoothness), **aspect ratio** and **absolute amount of fibres in the mix**.

The efficiency of the composite action between fibres and matrices is governed by the bond and by the contrast of elastic modulus between fibres and matrix.

➔ A good bond (bond/matrix cracking strength as high as possible) and a ratio $E_{\text{fibre}}/E_{\text{matrix}} \gg 1$ are key conditions.

➔ The bond must also not be too good to induce fibre breakage. Highly deformable UHPFRC can only be achieved with fibre pull-out mechanisms. Fibre breakage should absolutely be avoided.

UHPC matrix:

The major factors of influence on the performance of UHPC matrices (resistance, protective function, bond and workability for the composite) are:

- Packing density of grains
- Water/Fines – W/F ratio
- Degree of hydration of the binders α_{hydr} and confinement of hydration products
- Ultrafines/Cement² – U/C ratio and Ultrafines/Fines – U/F ratio
- Paste volume (% Vol.) or fine aggregate content
- Superplasticizer/Fines ratio – SP/F

Many different types of UHPFRC recipes with various matrices and fibrous mixes are currently under development worldwide. Very few or almost none however satisfy at the same time the conditions of tensile strain hardening, low permeability, high tensile and compressive strength and self compacting character needed for cast-in situ applications.

The trend is currently clearly to use local materials and by-products of the industry such as fly ash, Ground Granulated Blast Furnace Slag - GGBFS and combinations of them to replace cement. However, most often, the workability barrier linked to cement/superplasticizer compatibility issues remains an obstacle to the use of an efficient fibrous mix to achieve true tensile strain hardening and/or other drawbacks are encountered (higher shrinkage, limited availability of the materials, variability of the composition of the industrial by-products, high scatter of properties due to an insufficient fibrous mix).

A possible way to overcome this barrier is to replace cement grains by other particles of similar size and morphology but with a mineralogy providing a better compatibility with the plasticizers. Active such as Fly ash, latent active such as ground granulated slag, or inert particles such as quartz powder and limestone filler are good candidates for this.

Application to Slovenian and Polish components

The goal of the Research and Development works was to find recipes with the same fibrous mix, with comparable properties of Workability, Mechanical Performance and Protective

the "efficiency range" of the superplasticizer, a change of the dosage induces a significant change of the workability. The dosage at saturation is the one after which no more significant change in workability takes place.

² Cement is meant here as reactive clinker particles.

Function ("PMW" requirements) than the SAMARIS mixes, but using to the largest possible extend components available locally in Slovenia or Poland: Cement, Superplasticizer, Quartz Sand and Silica Fume.

A further goal was to improve the slope tolerance of specific mixes for cast on site applications on structures with slopes of 3 to 5 %.

Cements (Salonit Ahnovo – Slovenia and Gorazde - Poland) and Superplasticizers Zementol Zeta Super S® (TKK) for Slovenia and Sika Viscocrete for Poland were used.

First developments were started in Slovenia. Several attempts were made with Pure CEM I 42.5 Sulphate resistant and CEM I 52.5 R cement from SALONIT but with unsatisfactory workability despite high superplasticizer dosages. It rapidly turned out that UHPFRC recipes with such high fibre dosages and sufficient workability could not be achieved with local pure CEM I from Slovenia. The same trend was later confirmed for Polish products. Hence another way had to be found.

From there it was decided to investigate the possible replacement of large quantities of the cement used in the existing UHPFRC recipes from the SAMARIS project by limestone fillers. *The final outcome of those R&D works are three new UHPFRC recipe: for Slovenia recipes CM32_11 and CM32_13 and for Poland recipe CM33_9 with following properties:*

Self compacting character. Mechanical and protective properties equivalent to the mixes developed during the SAMARIS project, 50 % cement replacement by limestone filler.

- Recipe CM32_11 has limited slope tolerance but can be used to fill formworks with limited space.
- Recipe CM32_13 has a slope tolerance of at least 5 % but should be used only to fill open formworks of limited height (20 cm max.) and with sufficient space (30 to 35 mm minimum) if it is needed to avoid longitudinal casting joints between kerbs and bridge decks for example.
- Recipe CM33_9 has a slope tolerance of at least 3 %. This mix was validated in the laboratory on small scale batches (25 litres) and should be further optimized on larger scale trial tests.

Mechanical performance on the basis of flexural tests on small prisms and instrumented 4 PT bending plates (50 x 20 x 3 cm), representative of the application thickness, and protective function by means of air permeability and capillary water absorption tests were also investigated for those recipes, both at EPFL and ZAG and compared to the target values. All results are within the expected limits and no significant detrimental influence of the Thixotropizing addition could be observed

Trial tests were performed at the Salonit plant in October 2008 to verify and optimize in full scale the ability of recipes to accommodate slopes of 3 to 5 %. The test were successful and 900 litres of the new material CM32_13, with only 0.3 % Thixotropizing addition were applied from a concrete truck on two inclined test surfaces of 10 m² with 3 and 5 % slopes in the plant. The losses in the truck were extremely small (around 50 litres). Figure 35 shows the production and application of the UHPFRC.



Figure 35 Full scale field trial, Salonit plant, Slovenia, October 2008

Conclusions

- A methodology was proposed, validated and applied to develop local UHPFRC mixes from Slovenia and Poland, with a very large cement replacement by limestone filler.
- This concept also significantly reduces the monetary and environmental cost of UHPFRC, by decreasing to a large extent their cement content.

Both Slovenian recipes were used successfully at an industrial scale (total 15 m³ produced) during the first application of UHPFRC in Slovenia, for the rehabilitation of the Log Čezsoški bridge in July 2009.

All recipes satisfy the original requirements of using to the largest possible extent local products and have a potential to be further improved.

3.1.12 Full-scale applications Recommendations for the use of UHPFRC in composite structural members

Introduction

The increasing volume of European transport urgently requires an effective road and rail system in Central European and Eastern Countries (CEEC) with a major investment in building new and assessing and rehabilitating old structures.

Ultra-High Performance Fibre Reinforced Concretes (UHPFRC), characterized by a very low water/binder ratio, high binder content and an optimized fibrous reinforcement, provide the structural engineer with a unique combination of extremely low permeability, high strength and tensile strain hardening. UHPFRC are perfectly suited to the rehabilitation of reinforced concrete structures in critical zones subjected to an aggressive environment and to significant mechanical stresses, to provide a long-term durability and thus avoid multiple interventions on structures during their service life. Extensive R&D works performed during EU project SAMARIS and various full scale applications in Switzerland on bridges have demonstrated that UHPFRC technology is mature for cast in-situ applications of rehabilitation, using standard equipments.

EU Project ARCHES dedicates a significant effort to demonstrate the applicability of this innovative rehabilitation technique in CEEC, with cheaper UHPFRC based on locally available components and improved rheological properties (tolerance to slope of the substrate at fresh state).

Achievement of tensile strain hardening, extremely low permeability and self-compacting character is indeed a challenge that few current UHPFRC recipes can satisfy. An original concept of Ultra High Performance matrix with a high dosage of mineral addition has been developed that makes the application of UHPFRC technology feasible with a wide range of cements and superplasticisers.

In a further step, the rheology of those mixes has been adapted to enable them to accommodate challenging 5 % slopes of the substrates at fresh state. Finally, these new materials have been applied to the rehabilitation of a bridge in Slovenia.

The following document analyses this new application with innovative UHPFRC in the perspective of a sustainable use of construction materials. It also gives practical recommendations based on the experiences gathered during the site.

Rehabilitation of the Log Čezsoški bridge – Slovenia

The bridge is located in the very northwest of Slovenia, close to the city of Bovec, and crosses the Soča river, in a mountain region. It has only one lane and a frequent traffic as it is the only link between the two sides of the river within 15 km. The cross section of the bridge, with the concept of rehabilitation are shown on Figure 36.

- A continuous UHPFRC overlay with no dry joints is applied to protect the full upper face of the bridge deck, footpath and external faces of the kerbs.
- The thickness of the UHPFRC layer is varied according to the more or less difficult geometry to cast, and also in order to maximize the efficiency of the fibrous mix. The deck (A) has an overlay of 2.5 cm, the inner faces of the kerbs (B), 3 cm, the footpaths (C) 3 cm, as well as the external faces of the kerbs (D).

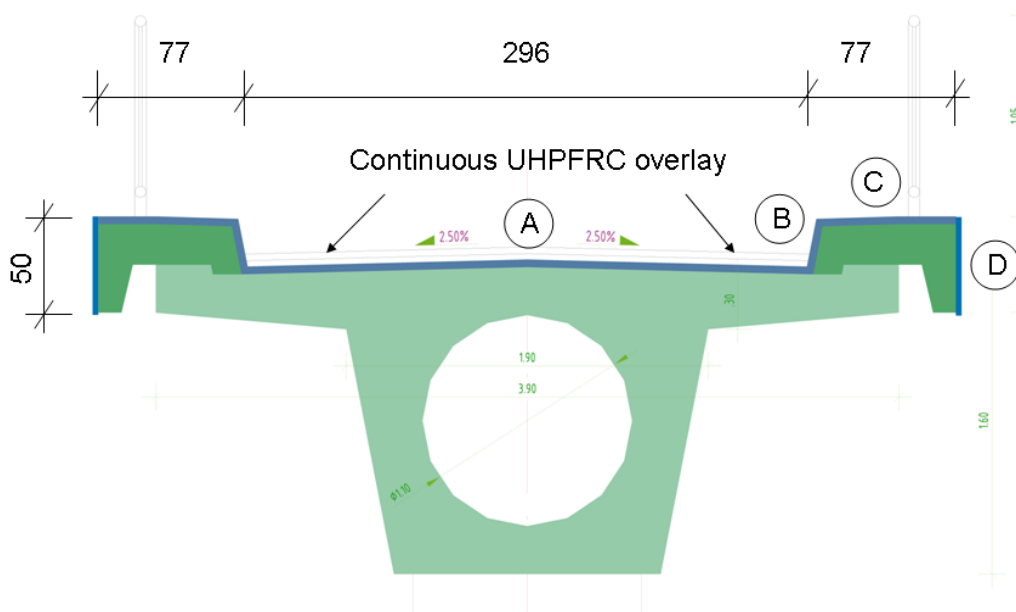


Figure 36 Cross section of the bridge with concept of rehabilitation, dimensions in cm.

The selected concept with no dry joints along the full cross section guarantees a continuous protection. However, it sets high requirements to the choice of the UHPFRC mixes:

- For parts (A), and (C): ability to hold the longitudinal and transversal slopes of 5 % and 2.5 %.

- For part (D), no slope tolerance needed but ability to fill properly the formwork over 50 cm height with a width of 3 cm.
- For part (B); most challenging, ability to hold the slopes and to penetrate in the narrow space of 3 cm of the formwork, without however completely flowing throughout it as the lower part of formwork has to remain open to guarantee the continuity of the overlay without any dry joint.

Following the requirements of the concept of rehabilitation, two new UHPFRC recipes, developed within the ARCHES project from products available in Slovenia, of the CEMTEC_{multiscale}® family, with different rheological properties, were used to satisfy the challenges of the site. New processing and surfacing techniques were also applied for the first time.

The materials were produced in a concrete plant, transported to the site by a truck, and poured directly from the truck into carts for the casting of the outer faces of the kerbs, or onto the bridge deck or open faces of the footpath. For each day of casting, the outer faces of the kerb were first realized with mix CM32_11. An inclined plate helped the workers fill the material in place. The material CM32_13 was then used to cover the footpath, fill the inner face of the footpath and finally cover the deck. A great care was taken to cover as fast as possible the fresh UHPFRC surfaces with a wet textile and a plastic foil, as external temperatures quickly reached 35 °C, Figure 37.



Figure 37 Processing of the materials on the site.

From a general point of view, one can say that the casting progressed well, as planned on two days on July 16 and 17, 2009, despite minor problems and that the workers very quickly took the UHPFRC technology into their hands, with standard tools. Dr. E. Denarié and Dr. Pierre Rossi were present for the application and together with Dr. Šajna advised the workers with the help of translators. The workability of the UHPFRC mixes over the full duration of the site was very satisfactory, despite some small incidents. The slope of 5 % was held without difficulties and the casting of footpath and outer faces of the kerbs went as expected.

The bituminous pavement was applied on the UHPFRC surfaces after 7 days of moist curing³, and the bridge was reopened to traffic just one month after the start of the works.

Owing to the special processing technique used for footpath (woods platens over

³ This moist curing is particularly important as the UHPFRC exhibits a very significant self desiccation at early age, and is prone to drying. After 7 days, this is no more the case.

ZEMDRAIN® foils), pedestrians and to some extent cyclists could use the bridge at the end of each casting day.

The overall surface appearance of the bridge after the rehabilitation is very satisfactory and barefoot walking is possible on the footpath. Several parts of the inner faces of the footpath were not filled properly with the UHPFRC and had to be filled later. An unsuccessful attempt was done to do this with a special UHPFRC mix and the decision was finally taken to fill those gaps with a high quality repair mortar adapted for this purpose.



Figure 38 The bridge after the rehabilitation.

Finally, a global assessment of the environmental impact of this system of rehabilitation was done. Four levels of assessment were analysed: 1: One cubic meter materials, 2: Effective material volumes per system, 3: All rehabilitation work involved, 4: All rehabilitation work considering the whole life cycle. Four systems were compared: two traditional rehabilitation systems and two rehabilitation systems using UHPFRC. The difference between the two solutions in the same system was the nature of the binder used.

➔ The impact due to the production of materials is the major contribution to the environmental impact of the rehabilitation. The UHPFRC that use local components has a similar impact than traditional rehabilitation systems using waterproofing membranes. Furthermore, if the durability of the rehabilitation is considered, this study shows that the impact of this innovative system is much lower than all the other rehabilitation systems as the durability of UHPFRC is much higher than usual concretes, Figure 39. Further, at a local level, a dramatically shortened site duration (by a factor 3) such as with the use of UHPFRC also helps decrease significantly the amount of detours from end users during bridge closure and thus the CO₂ footprint of the site.

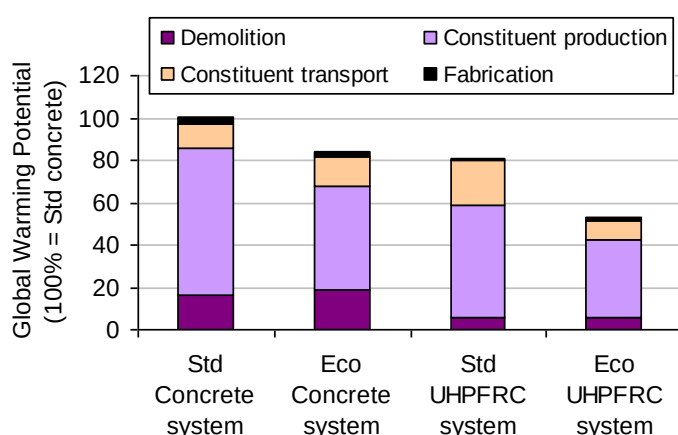


Figure 39. Global Warming potential induced by the different solutions for the Log Čezsoški rehabilitation, considering the life cycle. All solutions are compared to the traditional rehabilitation system with standard concrete taken as reference (100%).

Conclusions

- The concept of rehabilitation of structures with UHPFRC was applied for the first time outside of Switzerland, in Slovenia with a new material designed from local components.
- The application was successful and fast (1 month instead of 3 month with traditional technique) and demonstrated at an industrial scale the ability of the newly designed UHPFRC mixes to reply to the difficult challenges of the site.
- Applications with slopes up to 5 % at least are now possible, and by means of simple surfacing techniques it is possible to achieve uniform textured UHPFRC surfaces on which barefoot walking is possible.
- The newly designed recipes have a dramatically reduced cement content which makes them more economical and particularly attractive from an environmental point of view.
- This successful example of transfer of technology opens up very promising perspectives for the dissemination of the concepts of rehabilitation of civil infrastructures not only in NMS (which was the goal of the project ARCHES) but also in virtually any country.

Final Remarks

The Arches Project- Assessment and Rehabilitation of Central European Highway Structures has been accomplished within the European Commission 6th Framework Programme between the 1st of September and 31st of August 2009. As mentioned in the beginning of the Report all the Deliverables have been accomplished and at the moment belong to the end-users, who can take advantage of their advices. As the Project was intended to be disseminated in Central European countries some of those Deliverables will be translated onto national languages of the selected end users. This activity will be performed within the activity of another 6th Framework Programme CERTAIN. At the moment all the described Deliverables are available in English at <http://arches.fehrl.org/>