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Sustainable Development, Global Change and Ecosystems

Sustainable Surface Transport

Strategic Targeted Research Project

ARCHES

Assessment and Rehabilitation of Central European Highway Structures

Recommendations on the use of results of monitoring on bridge safety assessment and maintenance

Deliverable D08

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1. INTRODUCTION

1.1 General background

The ARCHES, which is the Specific Targeted Research Project, was planned in response to the European Commission's call for proposals 3B, addressing Topic 2.6 'Design and manufacture of new construction concepts' of objective 'Sustainable Surface Transport' under the Thematic priority 1.6 'Sustainable Development, Global Change and Ecosystems' of the GROWTH part of the Sixth Framework Programme. The contract was signed by the Commission on the 25th of October 2006. Project commencement date was the 1st of September 2006 and the duration of the project is 36 months.

1.2 Summary of project plan

The increasing volume of European transport urgently requires an effective road and rail system in Central and Eastern European countries (CEEC). To bring this transport infrastructure up to modern European standards will require an immense investment – and therefore difficult to achieve in the medium term. New motorways will be required with many new bridges. Numerous existing bridges will need to be assessed, and a large portion of them improved or replaced.

The overall goal of the proposed project is to develop ways to raise the standard of the highway structures of New Member States (NMS) and CEEC to the level necessary for their full economic integration into the EU and for the future development of the union.

The works were focused on the development of technologies and procedures for optimized conservation of highway structures, taking into account issues particular to the NMS. This can be achieved by a global conceptual approach which can be summarised as Avoid, Prevent and Harden. Rehabilitation and replacement will be as far as possible avoided by better safety assessment methods. Corrosion of reinforcement in concrete structures will be prevented with new concepts of localized cathodic protection, and the application of new, cheaper, low alloy steel reinforcement. Advanced materials with very low permeability and high ductility will be used to replace deteriorated concrete and waterproofing membranes, harden structures in critical zones and provide long-term durability.

The ARCHES project is part of a cluster of three complementary STREPs which will be coordinated by the CERTAIN coordination action. The purpose of this is to guarantee that Engineers in the field will be informed, trained and will implement the results immediately on completion of the research.

1.3 Objectives

The main objective of the structural assessment and monitoring part of the ARCHES project is to develop CEEC-appropriate techniques for optimal bridge assessment. 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. For example, SAMARIS (2006) has found that trucks in EU-15 and EEA (European Economic Area) countries are heavier than in NMS and CEEC – an average difference of 20% in traffic loading was found between sites in Netherlands and in Slovenia. Therefore highway structures in NMS and CEEC may have additional reserves in structural safety that, if known, can considerably reduce the required rehabilitation of bridges that have deteriorated.

Unfortunately, traffic loading conditions in NMS and CEEC highway structures *are mostly unknown*. The reason for such a situation is a lack of consistent systems that collect traffic data in a way appropriate for bridge design and assessment. 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. With that many unknowns, both on the loading and on the resistance sides, it becomes really difficult to obtain a reliable estimate of the actual bridge safety and serviceability and propose optimal rehabilitation measures.

Load testing of bridges has considerable potential to improve knowledge of load carrying capacity but is currently generally only practiced for new bridges and the information found is not used for assessment. 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. This deliverable describes how to use monitoring techniques for assessment, lifetime prognosis and in preventive maintenance. Another concept closely related to monitoring is the "soft load testing". During soft load testing, bridge's response to regular traffic is monitoring over a period of time. From this data, information about the bridge condition can be derived. The soft load testing concept was investigated within the ARCHES project and is thoroughly described in deliverable D16.

Gathering information on the actual traffic load on bridges and actual bridge condition is essential for improved management of highway structures. Using the advanced monitoring techniques available today (and being developed in projects such as *INTRO*) to gather the missing traffic load information and employing load testing, guidelines and Internet tools are provided which, if used, will avoid unnecessary interventions for a huge number of bridges.

The different subtasks integrating the Work Package, their interrelation and their dependence to the general framework of structural assessment is indicated in Figure 1.1.

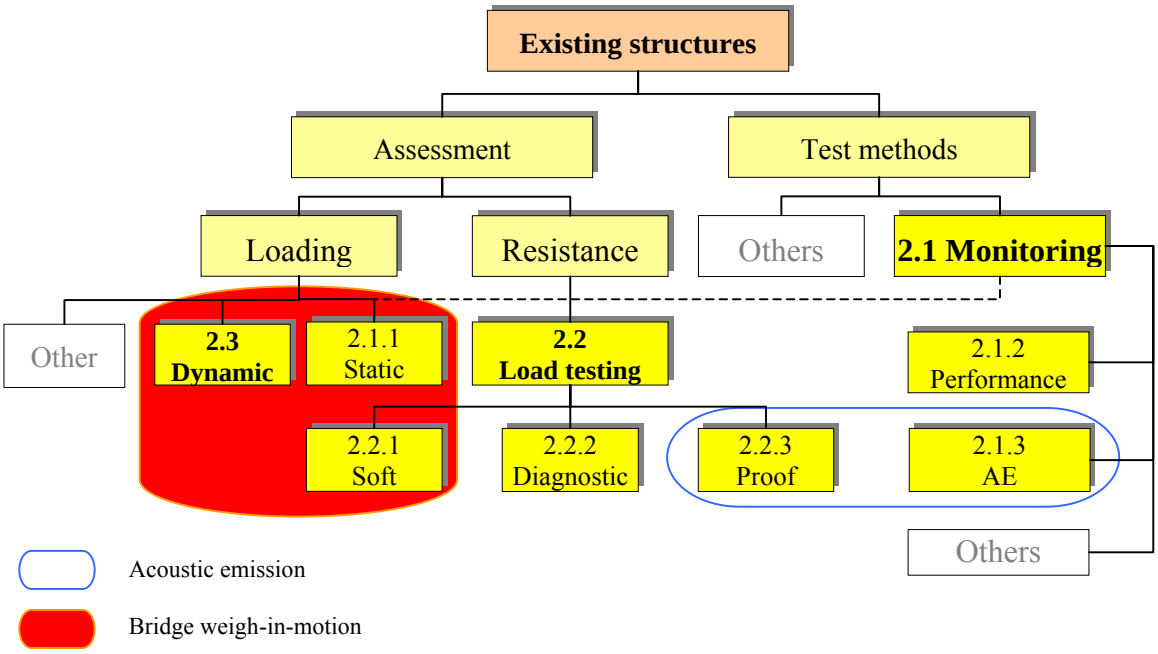


Figure 1.1 Subtasks of the work package

This report under the task "Measuring techniques and monitoring of structures" (2.1) is related to the existing sensors and their use in simultaneous monitoring in two directions: first to assess the real traffic loads on bridges (subtask 2.1.1), and second, to assess the actual bridge performance (subtask 2.1.2). In addition, a promising but rarely used monitoring technique, the Acoustic Emission (AE), will be incorporated into the non-destructive assessment process (subtask 2.1.3). This monitoring method was selected because of its potentials in proof load testing, where it is expected that it will indicate at an early stage, potential damage due to cyclic or incremental loading.

The subtask on *Bridge traffic load monitoring* (2.1.1) aims to quantify the increased traffic loading on bridges in NMS after EU enlargement. It combines available information and newly collected traffic load data (weigh-in-motion (WIM) measurements) to propose simplified assessment traffic load models at two levels: by taking into account the measured WIM data on a bridge of concern (a site-specific model) or by applying more generalised, but measured WIM data at the network level (road-specific model). Using both new types of assessment of the traffic load, it is anticipated that it will be possible, *in many cases, to prove sufficient structural safety* and prevent unnecessary or premature interventions in deteriorated bridges. This will greatly assist in the prioritization of repair/upgrading programmes throughout CEEC.

The subtask on *bridge performance monitoring* (2.1.2) describes the use of monitoring techniques for assessment of the actual condition of structures and lifetime prognosis. When performance is correctly evaluated, it will lead to increased service life of bridges and will prove higher structural safety. As monitoring, maintenance and safety assessment have generally been developing independently, the objective of this subtask is to correlate the monitoring results with the safety and maintenance issues.

The subtask (2.1.3) on *Acoustic Emission* (AE) investigates the correlation of cycle and incremental loadings with AE to assess the symptoms of bridge condition. Among the available non-destructive monitoring techniques, AE has been chosen for two reasons. First, it has been successfully used in the long-term monitoring of steel structures (buried tanks, pipelines,etc.). Despite its high monitoring and warning potential for progressive cracking of materials and some promising laboratory applications as well its feasibility and efficiency for long-term monitoring of steel structures, hindered by consumable

time and high costs, the AE has not found widespread applicability in long-term monitoring of existing concrete and steel bridges, yet. As the major advantage of this technique compared to the others is that it can alert against a progressive failure or warn when risk of failure is high, the AE has great potential in supporting the proof load tests.

The summary of the resulting recommendations on the use of structural monitoring for bridge safety assessment and maintenance is given in chapter 2. The main objective of these recommendations is to provide support to the assessment engineer on how to integrate monitoring techniques with maintenance methods and safety assessment tools. The available codes of practice and standards lack the information on how to correlate monitoring with real load carrying capacity or with assessment in general. Monitoring shows a potential to in the updating of existing models for deterioration processes and, as a consequence, time-dependent bridge safety and performance can be estimated more accurately.

2. USE OF MONITORING IN ASESSMENT

The re-assessment of existing bridge structures covers two major themes: assessment of traffic loads and assessment of structural resistance. Both themes are addressed in this document. Monitoring technologies can provide data, which can help to assess both loads and resistance more accurately. More accurate assessment may in some cases reveal reserves in structural performance, but on the other hand it may also in some cases reveal lower structural performance than it was expected. In both cases, the theoretical load and resistance calculations are brought nearer to reality by monitoring results.

Particularly, accuracy of following important information can be improved by monitoring results:

Loads
Effect of permanent loads
Traffic loads
Other variable loads and their effects

Resistance / Performance
Current static stress state
Traffic load effects
Structural changes in time

In monitoring-based assessment of a particular bridge, not all of the above listed factors have to be investigated by monitoring techniques. The list defines merely possibilities. The more information is extracted from monitoring, the more accurate becomes the final assessment. With increasing complexity of monitoring hardware and data evaluation, the system becomes expensive. However, there are possibilities of coupling several tasks. The hardware for traffic load monitoring also largely covers the needs of a structural performance monitoring system. This advantage can be used in designing one monitoring system that satisfies the data needs of traffic loads and structural performance monitoring.

Effect of permanent loads

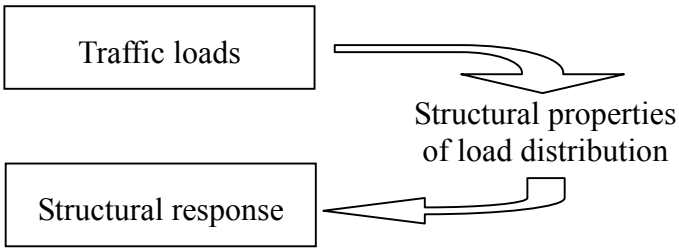
The effect of permanent loads is reflected in the current static stress state of the structure. The possibilities to measure stress state of existing structures are limited. Non-destructive methods exist for stress measurements of external tendons or cables. Stress in other structural parts cannot be measured non-destructively with current methods.

Measurements of stress in external tendons are recommended for evaluation of safety assessment. The magnetoelastic method is well developed and inexpensive. Accuracies of up to 1% can be achieved if material calibration is carried out. More information can be found in chapter B2.2.1, Annex B.

Reaction forces in abutments and piers can be measured by jacketing. The measured forces can be used to calculate force distribution of permanent loads in the bridge.

Traffic loads

A large part of the work was concentrated on traffic load and their effects. Traffic loads can be investigated in two points: the traffic loads itself, and influence of traffic load on the bridge. The two are connected by structural properties of load distribution.



If structural properties are estimated by a mathematical model of the bridge (e.g. Finite-Element model) and structural response is measured, the traffic loads can be identified. This principle is used in traffic load monitoring. The load monitoring measurements can be evaluated by a Weigh-In-Motion (WIM) algorithm that identifies static axle loads, or by Moving Force Identification (MFI) that tries to identify dynamic axle loads. The latter method requires an accurate Finite-Element model of the bridge. Therefore, it should be preferably used on bridges in good condition, of sufficient documentation. Preferred are bridge types without cracks.

The real loading condition of a bridge can be investigated by Weigh-In-Motion techniques, which are described in chapter 3.2. One result of Weigh-In-Motion are statistical distributions that describe the traffic flow. If it can be assumed that this identified traffic flow properties apply on certain part of the route, on which the bridge is located, then loading conditions on many bridges on that route are defined by results of just one traffic load monitoring system. Measurements have been carried out on motorways in 4 CEE countries. These results are presented here and can be used for traffic load assessment.

The traffic flow properties are used to calculate characteristic traffic load effects on bridges. The results depend on the bridge structure and have to be calculated for each bridge separately. The methodology of these calculations uses Traffic Load Modelling technique, described in chapter 3.3. Results for short to medium spans bridges and recommendations for the use of load models are presented in chapter 3.4.

If traffic loads are known (which is during a controlled test) and structural response is measured, the structural properties of real load distribution can be evaluated. This can be done by load tests. The results are very useful in evaluation of effects of prescribed load combinations on the bridge, since the real load distribution (described by influence lines) may reveal some structural reserves in comparison to results using theoretical influence lines. More information on this task can be found in Deliverable D16.

If only structural response is measured and evaluated, the actual effect of traffic loads on the measured bridge location is acquired. The knowledge of absolute traffic load values itself is not acquired, neither are structural properties of load distribution. More information on the use of this results can be found in chapter 4.2 and chapter B3.1.3, Annex B.

Other variable loads include environmental loads (temperature, wind, etc.). It is recommended to apply specialized sensors if continuous monitoring system is applied to measure the environmental conditions. This data can be used in processing of monitoring data with two different goals:

- to remove the effect of environmental loads from the measurements (data normalization),
- to extract the effect of environmental loads from the measurements.

Removal (or compensation) of environmental conditions from measurement results is required in tasks of damage detection. In here, the source of measured change of structural response has to be identified and it must be distinguished between environmental sources and structural change.

Extraction of effects of environmental loads can be used for example to identify stresses caused by temperature. Further the identified effects are used to build a model for their compensation.

More information can be found in chapter B3.2, Annex B.

Structural changes in time happen due to deterioration of the structure (chapter 4.3). The changes can be monitored by a monitoring system that provides information valuable for bridge maintenance. Structural deterioration can be observed by several signs (using proper hardware):

- load redistribution in the bridge (strain measurements)
- plastic deformation, crack growth (strain and crack measurements)
- stiffness decrease (eigenfrequencies)
- acoustic emission of active cracks (AE sensors)
- corrosion (corrosion sensors)

The acoustic emission is a relatively new and promising technique. Therefore, it was investigated separately from other monitoring techniques. More information on acoustic emission can be found in chapter 5 and Annex C.

Structural assessment can be improved by monitoring techniques. Unlike in the assessment of traffic loads, a monitoring system for structural assessment can only provide information about the bridge on which it is installed.

A variety of different monitoring setups exists. The recommended use of different monitoring techniques depends on the particular bridge and its current problem. There is no “one-size-fits-all” solution, but the application of the techniques described in this document has to be evaluated for each case separately. Available techniques and their capabilities have to be considered. Depending on the structural problem, monitoring systems can be designed to fulfil different goals (safety assessment, extension of service life). More detailed information can be found in chapter 4 and Annex B.

3. BRIDGE TRAFFIC LOAD MONITORING

This chapter presents the summary of work on bridge traffic load monitoring. More detailed information can be found in Annex A.

3.1 Introduction

Considerable progress has been made in the ARCHES project on Bridge Traffic Load Monitoring. This deliverable describes these developments which can be broadly divided into two types:

- Developments in Bridge Weigh-in-Motion and Moving Force Identification
- Developments in Traffic Load Modelling.

Bridge Weigh-in-Motion (WIM), first developed by Moses (1978), uses a bridge as a weighing scales. It uses measured strains on the bridge to estimate the static weight of a passing vehicle and its axles. There has been some progress since the concept was first developed and it is developed further here. Moving Force Identification (MFI) is a more ambitious approach. MFI uses measured strains on the bridge to estimate the dynamic forces being applied at each point in time as a vehicle crosses (Gonzalez, 2001). MFI has the potential to provide more information on the passing vehicle but the mathematics are much more complex and it may be some time before MFI results are implemented in practice. Nevertheless, it has considerable potential and developments achieved in the ARCHES project are reported here.

Traffic Load Modelling uses WIM data, collected using either an instrumented bridge or other technologies, to calculate the characteristic traffic load effects on the bridge. Many bridges are now routinely monitored and load effect values such as strains can be extrapolated to calculate characteristic extreme values for use in assessment. However, even with years of measured bridge load effects, the extrapolation process is unsatisfactory and is highly sensitive to local conditions. It has been found in ARCHES for a particular example, that the load effect trend changed after eight years, i.e., extremely rare types of loading scenario would be missed if results were extrapolated from less than eight years' data and the results would be wrong. Traffic Load Modelling can overcome this problem. WIM data is used to determine the necessary information about the vehicles – their weights, numbers of axles, axle spacings, inter-vehicle spacings etc. and this is used to simulate many decades of traffic loading on the bridge. The process is sensitive to the assumptions made but, when used well, can produce reliable calculations of characteristic load effects with 75 year or 1000 year return periods.

3.2 Bridge weigh-in-motion and moving force identification

Bridge Weigh-in-Motion (WIM) uses an instrumented bridge to calculate the axle weights of crossing vehicles. When the bridge is long and/or the axle spacings are short, it is difficult for the algorithm to detect which axle causes the measured strains. The result is that conventional

Bridge WIM systems have to incorporate several improvements and modifications of the conventional algorithm, which alone cannot be as accurate as they might be, particularly for closely spaced axles[1] on bridges with spans longer than 15 m. 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 ill-conditioning problem is addressed using a numerical technique known as Tikhonov Regularization (Tikhonov, 1977). This involves an adjustment to the equations which makes them less ill-conditioned. A trade off is necessary between too little adjustment which leaves the equations ill-conditioned and too much adjustment which changes the equations too much from the original true equations. In Chapter A2 (Annex A), this new innovation in Bridge WIM is described and it is shown to substantially improve the conventional bridge WIM algorithm.

The Regularized Bridge WIM algorithm is tested theoretically using dynamic simulations from a 4-axle vehicle bridge interaction model with a random road profile. In total 12 cases are analysed, two loading conditions (fully laden & half laden), three velocities (55, 70 & 80 km/hr) and two types of suspension system (steel sprung & air sprung). The static axle and gross vehicle weights are calculated using the new Regularized-WIM and the original Moses' algorithm. The results found with Regularized-WIM are generally much more accurate than those found using the original Moses' algorithm. The improvement is quite significant for axles within a group (3rd and 4th axles) due to the inability of the original Moses' algorithm to effectively separate weights of closely spaced axles. Table 1 summarises the results by both algorithms for the 12 sample runs (2 suspension types, 2 loading conditions and 3 speeds). It can be seen from this table that the mean error of single axles has been reduced from 20.8% to 5.5% and the mean error of axles within a group has reduced from 50% to 10.6%. In fact for all results the percentage error in axle weights is improved using the regularised solution. However the percentage error in the GVW is slightly disimproved over that of the conventional algorithm.

It should be emphasised that this chapter demonstrates the potentials for further improving the BWIM accuracy, which today, using other approaches, is already fully comparable and in some cases more accurate than other conventional WIM technologies.

[1] Despite this, Bridge WIM systems have been found to give accuracies which are as good as some of the best alternative WIM technologies

Table 3.1 Relative error statistics by original Moses' WIM algorithm and regularised B-WIM algorithm smooth profile

Criterion	number	Original Moses' WIM algorithm				Regularised Bridge WIM			
		mean error (%)	standard deviation (%)	max (%)	COST 323 Class	mean error (%)	standard deviation (%)	max (%)	COST 323 Class
Single axle (1st & 2nd)	24	20.8	16.9	63.5	E(55)	5.5	4.2	14.7	B(10)
Axle within axle group (3rd & 4th)	24	49.8	33.6	116.3	E(115)	10.6	7.0	28.6	D+(20)
Gross Weight	12	1.0	0.6	2.1	A(5)	1.9	1.0	3.7	A(5)

Moving Force Identification

While Bridge WIM is concerned with finding the static axle weights, Moving Force Identification seeks to find the dynamic forces applied by axles to a bridge as a vehicle crosses. This is a much more challenging problem as considerably more information is being sought from the same quantity of data. A great deal of literature has been published on the subject of MFI but most of it is theoretical and there have been few field studies. Inverse Dynamics is the background theory used for MFI. The forward problem is that of finding the strains in a bridge due to a crossing vehicle. The inverse problem on the other hand is that of finding the forces (from a passing vehicle) that induced some measured strains. Inverse theory, described in Chapter A1 (Annex A), uses three areas of mathematics. These are least squares minimisation with Tikhonov regularisation, dynamic programming which provides an efficient solution to the least squares problem and finally the L-curve method to find the optimal regularisation parameter.

Many previous MFI studies are based on 1-dimensional beam models. However, most shorter bridges have significant width and, to get accurate results, need to be considered as a 2-dimensional slab. A 2-D MFI algorithm is developed in ARCHES and described in Chapter A3 (Annex A). It allows for the transverse behaviour of the bridge and the effect of the transverse location of the truck. A finite element model of a simply supported orthotropic bridge is modelled using a plate element and modal decomposition.

It is difficult to test MFI algorithms as instrumented trucks which record applied forces to a bridge are not widely available. Even when they are available, such trucks generally only give the force at the level of the axle rather than the applied force at the level of the road surface. As a result, the 2-D MFI algorithm is tested here using a numerical approach. Strains on the

bridge are generated using an independently developed 3-D vehicle-bridge-road profile interaction MSc/NASTRAN finite element model, that is further contaminated with 2% Gaussian noise (to represent sensor error). Analysis is carried out on the accuracy of the algorithm for various velocities and road profiles. Figure 3.1 illustrates some typical results.

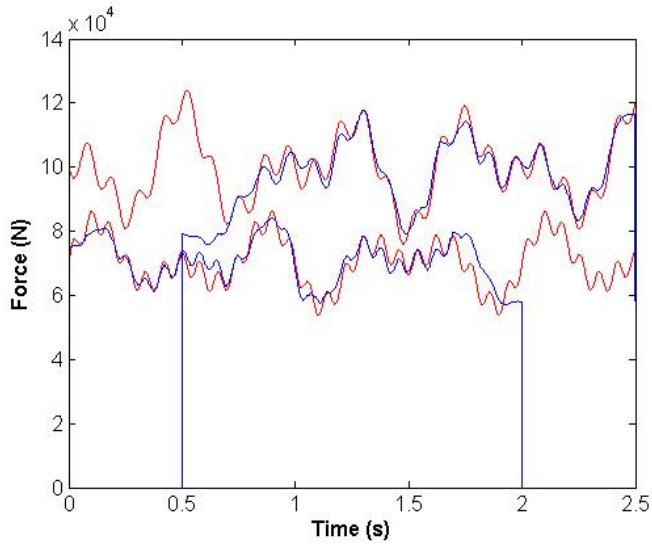


Figure 3.1 Identified axle loads using 3 sensors and 50 modes of vibration, (– theoretical force, – predicted force)

It can be seen in Figure 3.1 that the 2-D MFI algorithm is quite effective at capturing the variation in applied axle force as a 2-axle truck crosses a bridge. There are some inaccuracies, particularly as axles enter and leave the bridge, but the overall results are excellent. Not only are the major changes in force found as the truck bounces and rocks but the two major frequencies (axle hop and body bounce) are also accurately captured.

MFI has the potential to provide information unavailable from a Bridge WIM system, particularly relating to vehicle dynamics. However, today's most immediate problems are with assessing the loading on bridges in Eureopean New Member and Accession States. In this context, MFI has the potential to provide more accurate estimates of static axle weights. In other words, MFI can be used as an elaborate Bridge WIM algorithm. In ARCHES, field tests were carried out on the accuracy of MFI in estimating static axle weights. This work is described in Chapter A4 (in Annex A).

The experimental testing was carried out on the Vransko Bridge in Slovenia, a simply supported bridge of beam and slab construction. The axle weights were obtained by summing the identified forces in the middle 60% of the time history for each axle and averaging the result. These results were compared to the statically measured values for a test vehicle. The 3-axle test vehicle was monitored travelling in lane 1 of the bridge at 11.97 m/s. The identified force histories are illustrated in Figure 3.2. It can be seen that once all of the axles of the vehicle are on the bridge, the dynamic forces oscillate about their static axle weights.

The predicted static axle loads for this particular test are 68.6 kN, 203 kN and 272 kN, for the front axle, the sum of the rear axles and the gross vehicle weight respectively. The errors in the predicted axle weights are 7.90%, -1.54% and 1.03% respectively. The impact factors for

the identified force histories, can be defined as the ratio of the maximum applied dynamic force to the static force; the impact factors for this test run were found to be 1.13, 1.16 and 1.10 for the front axle, the rear axles and the total impact factor for the gross weight.

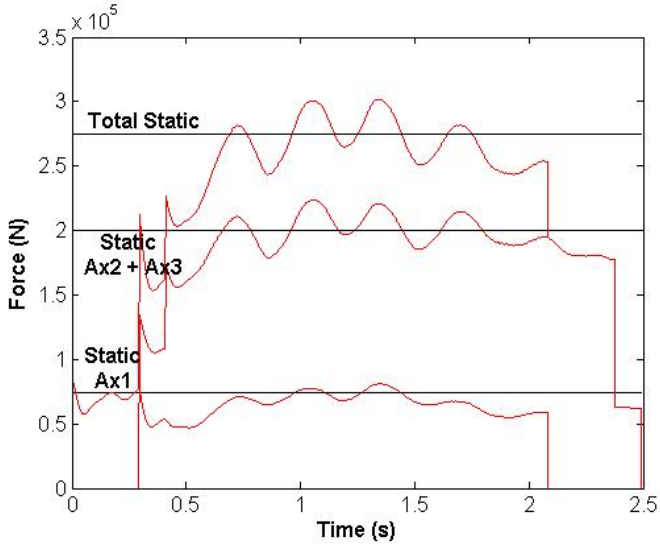


Figure 3.2 Identified forces for the test vehicle travelling in lane 1

3.3 Developments in traffic load monitoring

The Eurocode notional load model 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 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 the Dutch site, characteristic load effects were found to be 20% to 50% in excess of the levels suggested by the Eurocode. The vast majority of Europe's highway bridges are subject to considerably less traffic loading than this Dutch site. When designing new bridges, allowance should be made for economic (and hence freight traffic) growth and development, i.e., bridges should be designed conservatively, regardless of what country they are in. However, for existing bridges which in many cases have deteriorated over time, the issue is whether to intervene or not. 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. Hence, a major focus of this part of the ARCHES project has been on the development, for the first time, of a repeatable and reliable means of accurately calculating the characteristic traffic loading on an existing bridge.

The common approach to assessing 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 found in this way and uncertainty regarding the accuracy of fitted distributions are identified in Chapter A5 (Annex A)

as causes for concern. In this chapter Extreme Value Theory applied to bridge traffic loading is reviewed in the context of recent findings. The suitability of using random sampling and subsequent extrapolation to determine characteristic static load effects is assessed for a number of combinations of sample size and sampling period.

To test the accuracy of the extrapolation process, exact distributions for bridge load effect were obtained for some simple bridge loading events. For an example where only the gross weight is a variable, the exact distribution is found and compared with the results of conventional Monte Carlo simulation. An extrapolation from 25 daily maxima is repeated for a number of different 25-day periods. For the Generalized Extreme Value fits to the data, the characteristic 1000-year values vary in the range between –24 % and +33 % of the exact solution. It is concluded that small sample sizes and large extrapolation distances, as were used in the derivation of the Eurocode, provide highly variable results and should be avoided where possible. When the data is extended to a year (250 working days) of daily maxima, the situation improves. However, it is concluded that in general a short sampling period, i.e., a single day, does not have sufficient variability, and often may result in the mixing of incomparable loading event types. For example, consider the case of a bridge in which a single heavy vehicle constitutes the majority of daily maximum loading scenarios. However a more critical meeting of two or three heavy trucks may occur sporadically (e.g., once per year), in which case the extrapolated characteristic value is distorted by the lesser, daily maxima. Extrapolation of such samples should therefore be avoided in general.

Ten years of single vehicle events are simulated in Chapter A5 (Annex A), from which the 100 25-day maxima are obtained. When a Generalized Extreme Value distribution is fitted, the extrapolated characteristic value is approximately 3.8 % greater than the exact solution, while the Gumbel fitted distribution predicts a characteristic value of +4.5 % the exact solution. It is concluded that statistical extrapolation of load effects only becomes reliable when many years of traffic are simulated. This is necessary to generate the extremely rare combinations of vehicles that sometimes occur less than once per year.

Traffic Load Modelling for Central & Eastern European Countries

Given the variability in results from statistical extrapolation, techniques have been developed in ARCHES to enable very long-run simulations of traffic on bridges – up to 1000 years. Using importance sampling and parallel processing techniques, extrapolation has been avoided completely, thereby overcoming the uncertainty surrounding that process. Weigh-in-Motion (WIM) measurements were collected at WIM sites in Slovakia, Poland, Slovenia and the Czech Republic. The results are compared to data from the Netherlands as a reference.

Large sample sizes (hundreds of thousands of vehicles in each case) allow for more accurate estimation of the underlying statistical distributions and for a better comparison of observed and simulated daily maximum load effects. 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 simulation model presented here encompasses all the observed vehicle types but also allows for the probability that, even without freight growth, vehicles heavier and longer than those observed will be seen during the design life of a bridge.

The study is focussed on short to medium span bridges up to 45 m long for which free-flowing, rather than congested, traffic is considered to be critical.

The ten heaviest vehicles in the Netherlands and in Slovakia are shown in Table 2. They can be broadly classified as either cranes/crane-type vehicles or low-loaders with a maximum axle spacing of 10 to 12 m.

Table 3.2 Ten vehicles with highest GVW by site

(a) The Netherlands (A12)

No.	GVW (t)	No. Axles	Wheel-base (m)	Speed (km/h)	Maximum Axle Spac-ing (m)	Average Axle Spac-ing (m)	Moment on 45 m span (kNm)
N1	166	12	28.7	78	11.0	2.6	10 347
N2	165	12	27.3	85	10.6	2.5	10 743
N3	152	13	28.4	80	10.5	2.4	10 381
N4	150	12	28.8	79	11.1	2.6	9 685
N5	148	13	19.5	76	2.8	1.6	12 797
N6	147	12	28.8	81	11.1	2.6	9 847
N7	145	11	24.8	82	11.2	2.5	9 997
N8	145	13	29.4	80	10.5	2.5	9 211
N9	143	12	28.8	77	11.1	2.6	9 297
N10	140	13	28.3	84	10.4	2.4	9 253

(b) Slovakia (D1)

No.	GVW (t)	No. Axles	Wheel-base (m)	Speed (km/h)	Maximum Axle Spac-ing (m)	Average Axle Spacing (m)	Moment on 45 m span (kNm)
S1	117	9	26.1	94	6.0	3.3	7 238
S2	114	6	12.5	100	3.3	2.5	8 391
S3	109	8	14.0	44	2.7	2.0	9 816
S4	108	9	31.6	117	6.4	4.0	5 939
S5	107	11	27.6	45	11.4	2.8	7 052
S6	104	9	25.8	95	6.0	3.2	6 420
S7	101	11	18.5	44	4.7	1.8	8 854
S8	100	8	14.7	106	4.3	2.1	7 104
S9	99	8	19.2	64	10.6	2.7	5 936
S10	95	11	26.6	72	10.0	2.7	5 316

These extremely heavy vehicles might had special permits and escort vehicles, but were recorded travelling at speeds similar to other traffic and are typically part of the general traffic on the highway.

The parameters for each individual truck, and for the arrangement of trucks in each lane, are generated using statistical distributions derived from the observed traffic. The 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. A typical result is illustrated in Figure 3.3 where 8 years of simulated data is compared to 290 days of measurements in Slovakia. Five event types are shown – the 1- and 2-truck same-lane events, the 2-truck meeting event (with one truck in each lane), the 3-truck meeting event (with two trucks in one lane), and the 4-truck meeting event with two trucks in each lane.

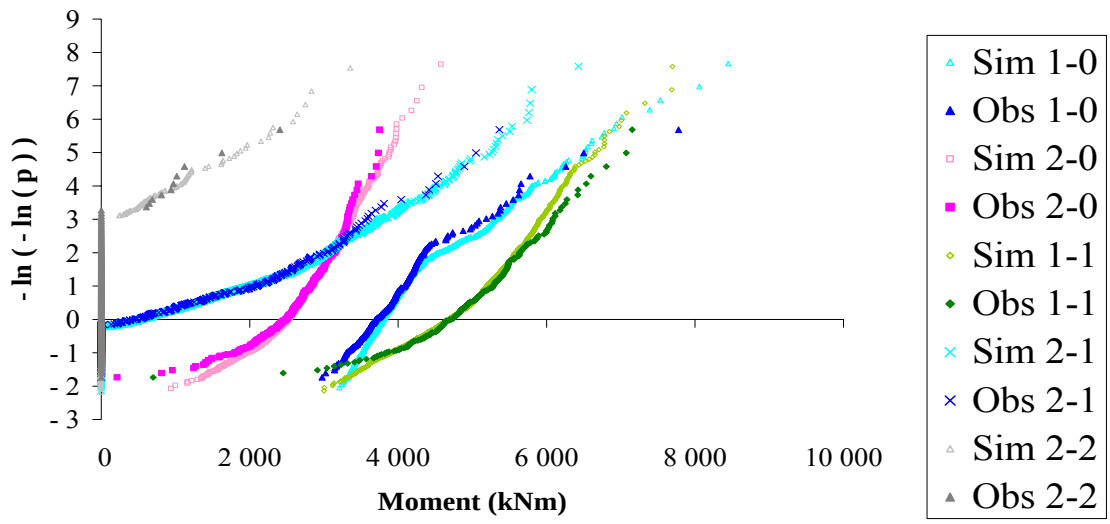


Figure 3.3 Maximum daily mid-span moment in simply supported, 35 m long bridge, Slovakia, bi-directional traffic (*i-j* = *i* truck(s) in lane 1 and *j* in lane 2)

Optimization is achieved 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.

Simplified traffic load modelling procedure

Results of the very exact simulation method to evaluate the projected live load effect were compared with results of a much simpler *simulation* method known as *convolution*. This 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 meters. Such bridges comprise well over 90% of all bridges in Europe.

The response of the bridge is associated with the arrival probability of vehicles in adjacent lanes and the probabilities of their gross weights, axle loads and axle configurations of the trucks. The probabilities are then assembled in cumulative distribution histograms for one loading event $F_M(m)$. By using the concept of extremal distributions, the probability distribution of the maximum response in a time period is written as a function of number of events, N, where N is number of the multiple-presence events, with several vehicles on the bridge, in a specified time period.

Figure 3.4 displays distribution of bending moments calculated for Vransko bridge, that was used to calculate the maximum expected moments and shears with the convolution method. Figure 3.5 depicts results of convolution for different time periods. Similar results are calculated for shear or hogging moments.

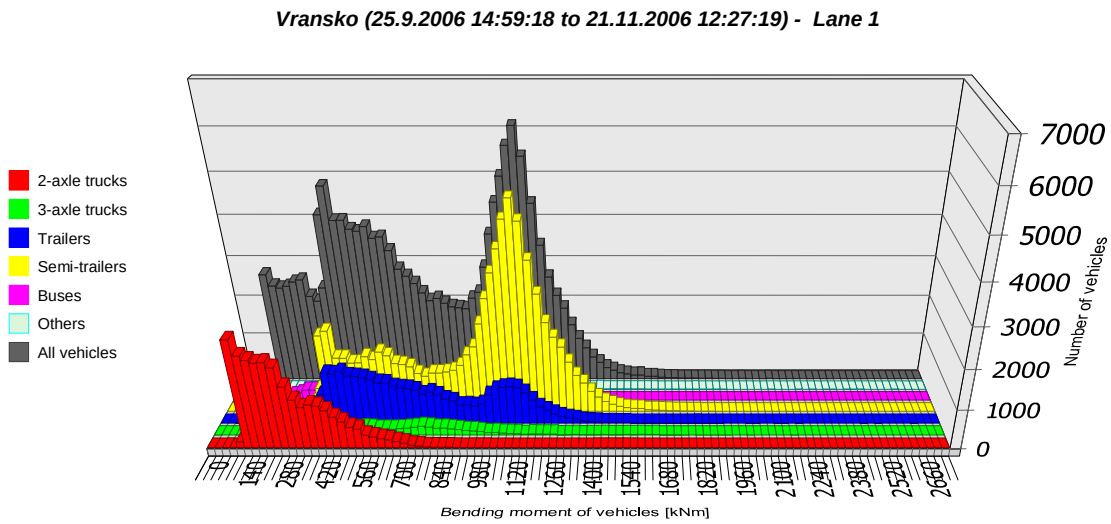


Figure 3.4 Bending moment histograms from Vransko bridge used for convolution

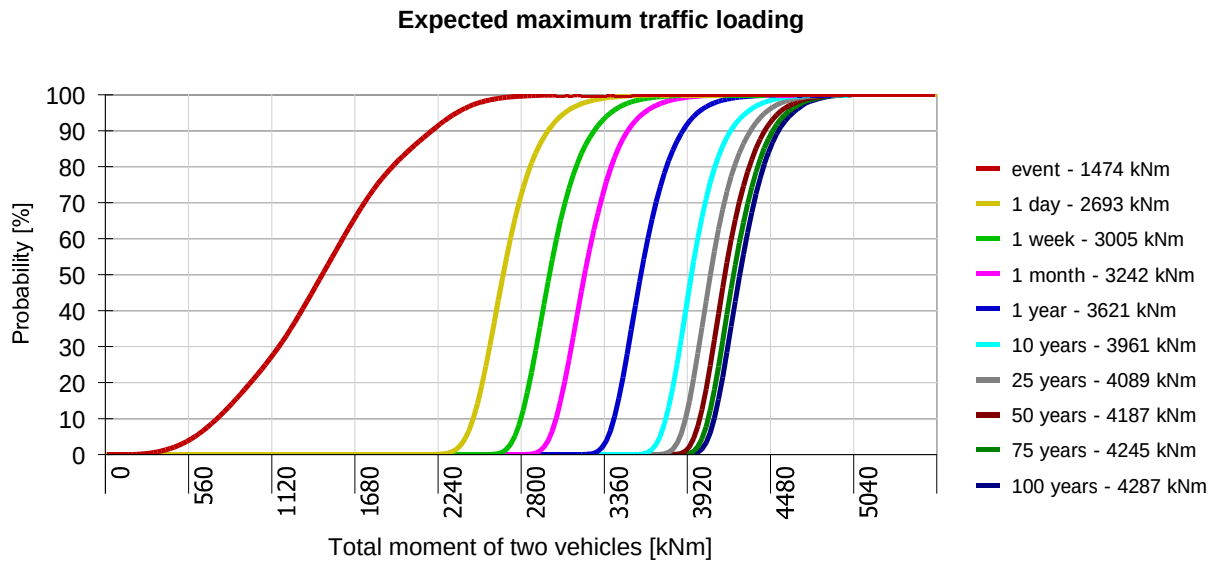


Figure 3.5 Expected maximum bending moments in different time periods (as a function of the number of events N)

In order to compare results of convolution with the new most advanced simulation method, the convolution method was also applied on the simply supported bridges with spans of 15, 25, 35 and 45 meters. Results are compared in Table 3.3, above (bending moment) and below (shear). They exhibit that, except for the 15-m span, where convolution results are 7% higher then those of the new simulation method (more conservative, but very close to the high lane factor value), the results of bending moments for both methods are on average within 1.5% of each other. Differences is slightly bigger for shear, where the convolution gives for the 15-m span 16.5% higher (more conservative) values, and for the other spans on average 9% higher values than the new simulation method.

Table 3.3 Comparison of the 50-year and 75-year bending moments (above) and shear (below) obtained by the new simulation method and the convolution method

	50-year						75-year					
	New method		Convolution		New vs. Convol.		New method		Convolution		New vs. Convol.	
Span	Mean	STD	Mean	STD	Mean	STD	Mean	STD	Mean	STD	Mean	STD
15	2.683,5	5,85%	2.910,0	3,35%	**-7,78%**	74,83%	2.733,5	5,75%	2.949,0	3,39%	**-7,31%**	69,40%
25	5.646,5	6,00%	5.652,0	2,97%	**-0,10%**	102,01%	5.750,5	6,00%	5.710,0	3,18%	**0,71%**	88,79%
35	8.975,0	6,30%	8.825,0	2,61%	**1,70%**	141,57%	9.123,0	6,40%	8.910,0	2,77%	**2,39%**	130,74%
45	12.572,5	5,80%	12.513,0	1,97%	**0,48%**	193,71%	12.780,0	5,80%	12.617,0	2,18%	**1,29%**	166,40%

	50-year						75-year					
	New method		Convolution		New vs. Convol.		New method		Convolution		New vs. Convol.	
Span	Mean	STD	Mean	STD	Mean	STD	Mean	STD	Mean	STD	Mean	STD
15	712,0	5,85%	860,0	2,81%	**-17,21%**	107,89%	726,0	5,75%	870,0	2,83%	**-16,55%**	102,94%
25	876,0	6,00%	988,0	2,68%	**-11,34%**	124,21%	891,0	6,00%	998,0	2,78%	**-10,72%**	115,47%
35	998,0	6,30%	1.093,0	2,15%	**-8,69%**	193,27%	1.015,0	6,40%	1.103,0	2,37%	**-7,98%**	170,57%
45	1.096,0	5,80%	1.215,0	1,43%	**-9,79%**	305,93%	1.115,0	5,80%	1.225,0	1,30%	**-8,98%**	345,45%

The convolution method was only compared to the new simulation method on Vransko bridge which prevents from giving firm conclusions about its accuracy. It however confirms that it 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 new simulation method. It is however not appropriate for longer bridges and for bridges with more than 2 traffic lanes.

Effect of influence line

Shape of the influence line has an important influence on the results. In calculations above the theoretical influence lines were used. If on the 24.8-m Vransko bridge, which is a 10-year old simply supported bridge with *rubber bearings* and *no expansion joints*, these are replaced with the experimental influence lines, obtained by updating the structural model to match the response of the measured structure (Figure 3.6, above), the bending moments drop considerably (Figure 3.7). As a result, the 75- year mid-span bending moments decrease for 42%. While similar benefits can be expected for the hogging moment, the experimental shear influence line always gives higher values than for a theoretical simply supported span (Figure 3.6, below). Yet, the differences are much smaller and have on the Vransko bridge reached 3.6%.

Experiences with soft load testing (ARCHES D16) also indicate that differences between theoretical and experimental influence lines for bending moment can be especially large on shorter and older bridges, where boundary conditions are not known. In such cases soft load testing can provide the necessary data to optimise bridge assessments.

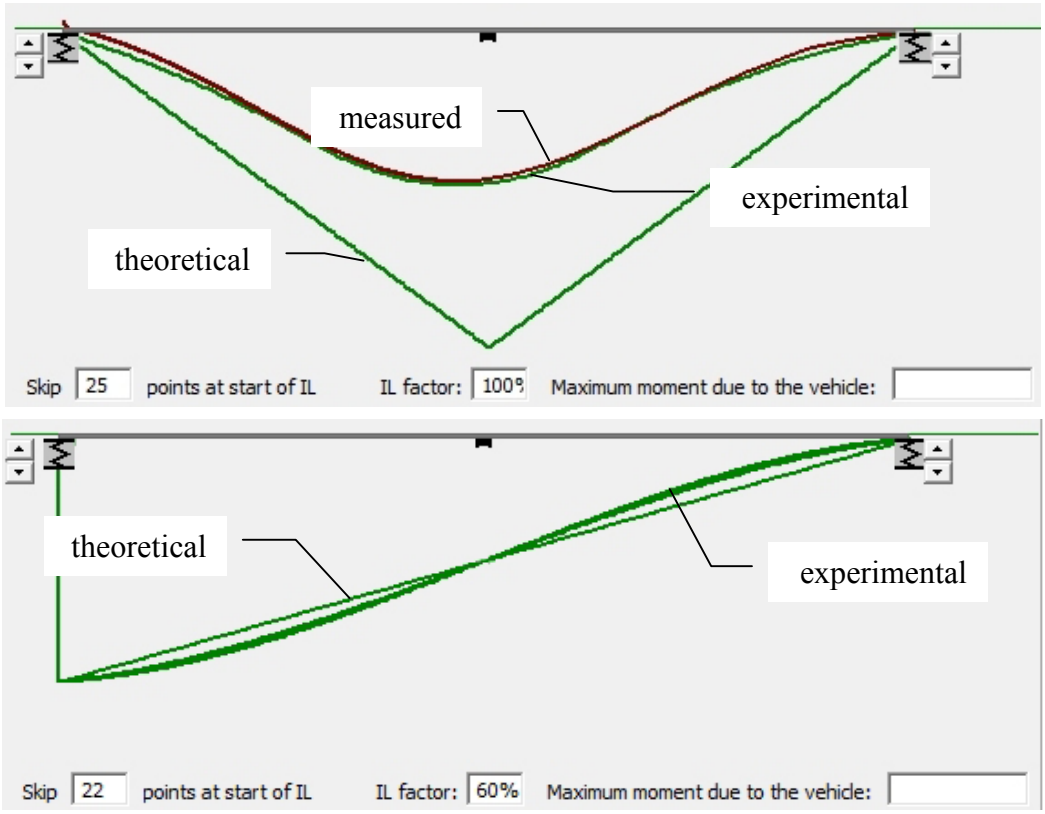


Figure 3.6 Influence lines for bending (above) and shear (below) on Vransko bridge

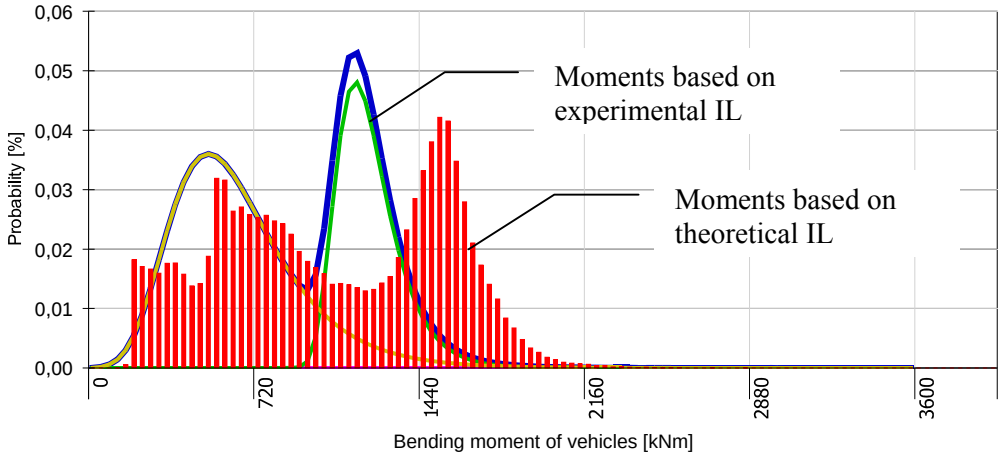


Figure 3.7 Comparison of bending moments calculated from theoretical and experimental influence lines in driving (above) and overtaking lane (below)

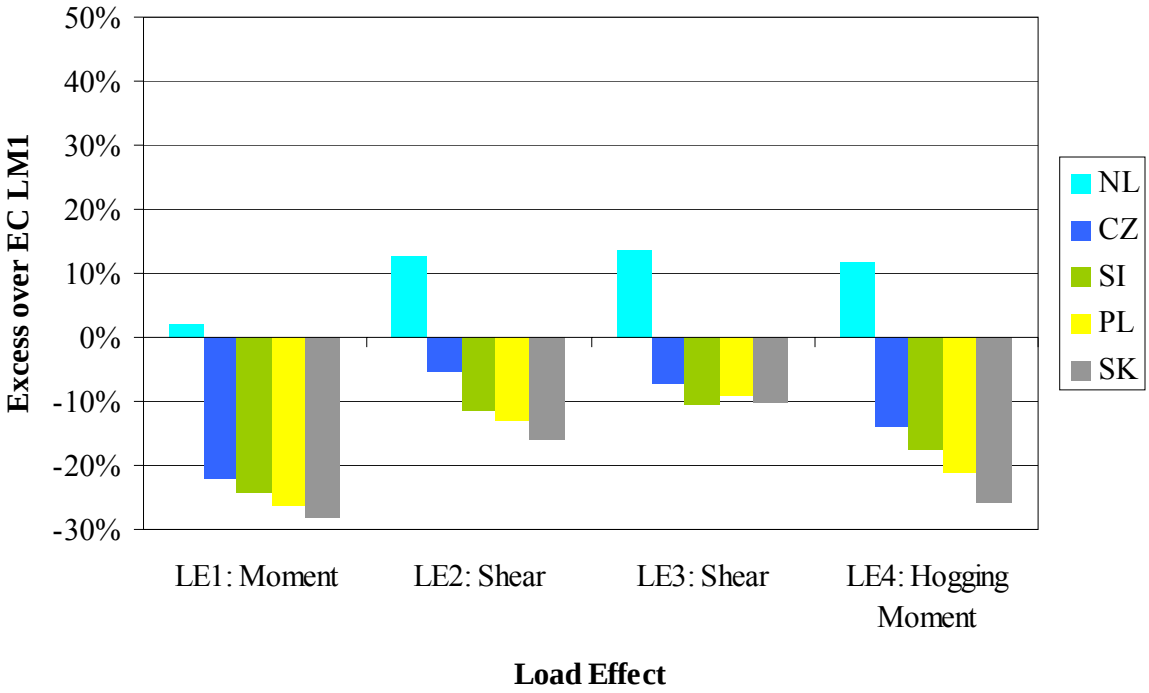
Knowing the real behaviour (influence lines) of a bridge has important consequences for optimised bridge assessment. While using theoretical simply supported influence lines provides important reserves for design of new bridges that can be used if in the future their condition deteriorates or the traffic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

3.4 Results

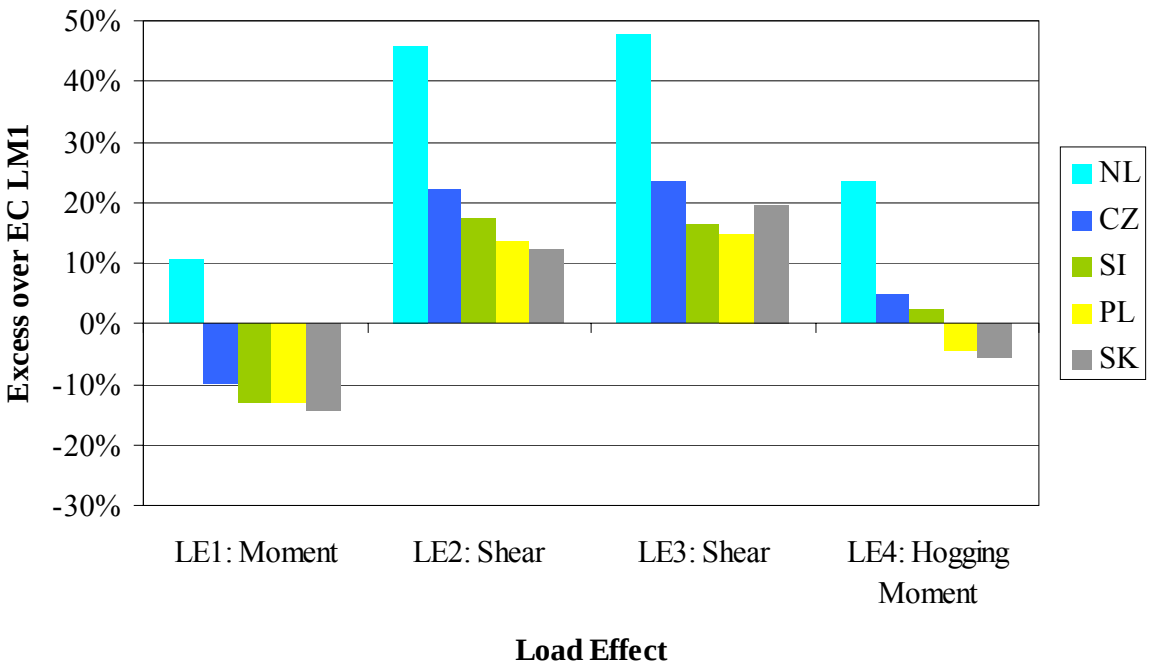
Results are compared to the Eurocode Normal traffic load model, LM1, for a wide range of load effects, spans and sites. If the structural element is between lanes then each lane will contribute equally to its stress and the 'lane factor' is defined as unity, i.e., the second lane contributes 100% the same as the first. 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. Results were calculated for the two extremes of lane factor. Values for lane factor are given in Figure 6.14 of Annex A.

Load effect values are found to be highly sensitive to this factor and it was found in ARCHES that the Eurocode load model LM1 is not a good representation of real traffic load in situations where lane factor is particularly low.

For four load effects, the Eurocode was compared with the results from the simulation model for each site, and for each site the maximum difference from four bridge lengths, 15m, 25m, 35m and 45m, are shown in Figure 3.8. The variation between bridge lengths at a given site is quite small.



(a) High lane factors



(b) Low lane factors

Note: At each site, for each load effect, the span with the highest ratio to the Eurocode is used for these charts. Dynamic effects have been removed from Eurocode LM1 – comparison is between static load effects

Figure 3.8 Comparison of characteristic loads with Eurocode LM1

Characteristic load effects for the Netherlands site can be seen to be up to 50% in excess of the levels suggested by the Eurocode. However, the Dutch traffic data was collected from a motorway site and the load effects were calculated for bi-directional traffic, as would be found on a less busy road. The numbers of trucks at the Dutch site were far in excess of levels that would be expected at any non-motorway site. The truck traffic on the motorways is in fact so great that congestion would be likely if it were applied outside of a motorway. In such a case, the dynamic amplification factors would no longer apply. The value of the Dutch data in this report is that it provides evidence on the nature of extremely rare and extremely heavy vehicles.

The four Central European countries have fairly similar results, and are lower than the Netherlands, but there are still significant excesses – up to about 20% – over the Eurocode, for bridges with low lane factors. As for the Netherlands, motorway data was used in these calculations for bridges subject to bi-directional traffic. However, in the case of the four new member state sites, 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 considerably less conservative than previously thought and will become less so as the frequencies of extremely heavy vehicles increase, as they undoubtedly will (Figure 3.9). 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 within the acceptable range.

Figure 3.9 Extreme Vehicles on European roads: crane-type (left) and low-loader (right) vehicle

Eurocode Load Model 3

Eurocode Load Model 3 ("LM3") provides for a set of standardised vehicle models (permit vehicles), assumed here to be travelling at normal speeds, which is the case for the measured traffic. For each site and bridge span, the characteristic load effects found from the simulations are compared with the load effects produced by each of the standardised models of LM3, and the model with the minimum gross weight that produces a load effect greater than or equal to the characteristic value for all spans is selected as the "required LM3". The results of this analysis are summarised in Table 3.4 by individual load effect and in Table 3.5 for all load effects. As can be seen, the load model required is typically governed by shear.

Table 3.4 Minimum LM3 model required for each site and load effect

Site	Lane Factors	LE1: Mid-span moment		LE2: Shear at entrance		LE3: Shear at exit		LE4: Hogging moment	
		LM3 Required	LM3 Excess	LM3 Required	LM3 Excess	LM3 Required	LM3 Excess	LM3 Required	LM3 Excess
Netherlands	High	1200	14%	1200/200	0%	1200	5%	1200/200	2%
	Low	1200	0%	1800/200	4%	1800	8%	1500/200	3%
Czech Republic	High	900	20%	1200	17%	1200	23%	1200	25%
	Low	1200	24%	1500	3%	1500	9%	1200/200	2%
Slovenia	High	900	23%	1200	21%	1200	26%	900	1%
	Low	900	2%	1500	2%	1500	9%	1200/200	0%
Poland	High	900	26%	900	3%	900	3%	900	11%
	Low	900	8%	1200/200	1%	1200/200	1%	1200	8%
Slovakia	High	900	32%	900	7%	900	4%	900	18%
	Low	900	11%	1200	0%	1200/200	1%	1200	9%

These tables show that the Eurocode load model LM3 can be conservative for the assessment of bridges subject to these traffics for current traffic conditions, if the National Application Documents for these countries specify the vehicles identified in Table 3.5. However, if this 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. Given that this is unlikely, National Application Documents for the design of new bridges should specify the 1800/200 vehicle (in both lanes). This level of loading is considerably greater than that being specified in most countries at the present time.

Table 3.5 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	1200
	Low	1500
Poland	High	900
	Low	1200/200
Slovakia	High	900
	Low	1200/200

Recommendations for Assessment

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 associated with shorter periods between consecutive inspections or assessments, can be justified for the assessment of existing structures. While new bridges in Europe are designed for a characteristic traffic action corresponding to a return period of 1000 years, the ARCHES group recommends a characteristic value of the traffic action for assessment corresponding to a return period of just 50 years, allowing lower safety levels for existing structures.

In this study, the WIM data was collected from motorway sites. The key elements for bridge loading are the weights and frequencies of the extreme vehicles (cranes and low-loaders). 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. The ARCHES report gives tables which show the reductions that this results in for load effects in each of the countries considered.

Table 3.6 Reduction factors for reduced truck volumes and for assessment

Truck volume as % of site ADTT[a]	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

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

4. BRIDGE PERFORMANCE MONITORING

This chapter contains a summary of results on bridge performance monitoring. More detailed information can be found in Annex B.

4.1 Introduction

The topic of operational monitoring is already well established nowadays and was implemented on a number of bridges worldwide (see chapter B5, Annex B). However, the issues of diagnosis, prognosis and maintenance are still under development today. The goal of the task on *Bridge Performance Monitoring* was to give recommendations on the use of monitoring for diagnosis and maintenance-relevant issues.

The present monitoring techniques have been reviewed. Three major application areas for monitoring systems have been identified:

- Safety assessment: Monitoring techniques are used to assess the reliability or resistance of the bridge.
- Extension of service life of deteriorating bridges: Monitoring techniques are used to observe development of known defects. Measured values are compared to predefined limit thresholds.
- Extension of service of bridges of special importance: Monitoring techniques are applied to monitor structural integrity of bridges in good condition.

The application of monitoring can be planned for 3 types of time periods:

- Short-term monitoring is a one-time application. Typical monitored time periods are 1-7 days. It is suitable for determining absolute structural parameters (e.g. stress in external tendons, eigenfrequencies, etc.). The results can be used to verify structural design assumptions. It has **limited applications for safety assessment**. The influence of environmental changes cannot be sufficiently evaluated. Structural response to traffic loads for safety assessment can only be determined by a test with controlled load. Recommendations on load tests are in Deliverable D16.
- Periodic monitoring is a repeated short-term monitoring application. Similarly to short-term monitoring, the results can be used to verify structural design assumptions, with the same limited applications for safety assessment. Additionally, development of the monitored quantities (stress, cracks, etc.) can be observed. It can be applied for **monitoring of deteriorating bridges**. The influence of environmental changes cannot be sufficiently evaluated, which is a **limiting factor for accuracy** of the results.
- Long-term monitoring is a permanent (>1 year) installation of the system. The system allows a good evaluation of environmental changes and also of history of traffic loads. Large amounts of data are delivered, which is useful for statistical

evaluation and accuracy estimation. Long-term monitoring is recommended for all types of monitoring applications.

The recommendations in this document are focused primarily on the use of long-term monitoring systems.

4.2 Safety assessment

Monitoring techniques can be used for safety assessment of bridges. Several techniques exist, which are dealing with this topic. Every technique has different limitations and probably none is able to give complete knowledge of structural condition. However, the knowledge extracted from monitoring results is a distinct improvement compared to the theoretical assumptions of the computational model of the bridge.

The techniques usable for safety assessment can be organized as follows:

```
Safety assessment
├─ Long-term monitoring
│   ├─ Reliability of structural components
│   │   ├─ Traditional assessment with correcting information from monitoring
│   │   └─ Fatigue failure mode
│   │
│   └─ Reliability of the bridge
│
└─ Short-term monitoring
    └─ Load tests
```

Particular techniques are described in the text below.

Long-term monitoring is performed with permanent installed monitoring system.

Advantages are:

- Environmental changes can be evaluated and compensated
- History of loads is recorded
- Delivers many data, which allows application of statistical methods and uncertainty estimation

Disadvantages are:

- Hardware cost
- Extensive data processing due to large data amount
- Automated system is necessary

Reliability of structural components determines risk of material failure at certain locations on the bridge. The assessment is connected to evaluation of stresses at these locations. Basic step for designing the monitoring system is selection of locations on the bridge that are to be monitored. Risk analysis should be performed and locations, where damage is most likely to occur, must be selected.

The selected locations are equipped with sensors. Typically long-term stable strain sensors (vibrating wire or fibre optics) are applied. Vibrating wire sensors are preferred due to costs. In case of external tendons, elastomagnetic sensors or vibration measurements can be used to determine material stress. The measurements are evaluated using the following techniques.

Traditional assessment with correcting information from monitoring

Traditional assessment is the procedure of theoretical assessment of Ultimate Limit State according to the Eurocode. Monitoring techniques can be used to adjust the theoretical loads to the real measured loads. Particular load components (dead load, traffic load, etc.) have to be expressed in terms of stresses at monitored locations to allow direct comparison with monitoring values.

Theoretical permanent load actions can be replaced

- if absolute stresses can be measured (e.g. at external tendons) by:

$$\gamma_G \cdot G_k + \gamma_P \cdot P \rightarrow \gamma_m \cdot \sigma_m \ , \qquad \text{(Eq. 4.1)}$$

where σ_m is measured material stress,
γ_m is partial factor to account for measurement accuracy. It should represent 95% confidence interval of the measurement

- if only relative stress changes can be measured (in most applications) by:

$$\gamma_G \cdot G_k + \gamma_P \cdot P \rightarrow \sigma_{0,d} + \gamma_m \cdot \Delta\sigma_{m,st} \ , \qquad \text{(Eq. 4.2)}$$

where $\sigma_{0,d}$ is design value of estimated dead load stress at beginning of monitoring,
$\Delta\sigma_{m,st}$ is static part of measured stress changes.

The permanent load actions adjusted by monitoring results contain all slow-changing actions, which includes structural deterioration, load redistribution in the structure during monitoring, but also temperature loads.

Traffic load actions (γ_Q . Q_k) can be adjusted with monitoring results by measuring influence lines in a controlled loading test. Measured influence lines can then be used to recalculate traffic load actions using traffic load models defined in the Eurocode. The measurement of influence lines by soft load testing is not a part of this document, but more information on this topic can be found in Deliverable D16.

This method is recommended if either
- absolute stresses can be measured – which is on external tendons, or if
- relative stress changes are expected due to structural deterioration.

Fatigue failure mode

The estimation of fatigue accumulation can be done with relatively low effort from stress (or strain) measurements. The stress cycles are counted using the rainflow method.

Fatigue damage factor can then be calculated:

$$d_t = \sum_{i=1}^{m} \frac{n_{\Delta\sigma,i}}{N_{\Delta\sigma,i}} \quad , \qquad \text{(Eq. 4.3)}$$

d_t is the fatigue damage factor of damage accumulated during measurement time t,
$n_{\Delta\sigma,i}$ is the quantity of measured stress cycles with the amplitude $\Delta\sigma$,
$N_{\Delta\sigma,i}$ is the quantity of allowable cycles with the amplitude $\Delta\sigma$ according to the respective S-N curve defined in the Eurocode.

If it can be assumed that the stress cycle histogram will not change during bridge lifetime, the remaining lifetime with respect to the fatigue failure mode can be calculated as:

$$t_{rem} = \frac{t}{d_t} - t_{past} \quad , \qquad \text{(Eq. 4.4)}$$

t_{rem} is the remaining lifetime
t_{past} is the elapsed lifetime.

In the case where other than constant traffic volume prognosis is present, the bridge lifetime can be split to 1-year intervals. In each interval, the stress histogram has to be re-evaluated according to the traffic volume prognosis.

One very new approach to assess capacity of structural component was published by Frangopol (2008). The method uses strain measurements to calculate probability of material strength exceedance at the measured location. Because the method is based solely on measurement data, real load distribution in the bridge and real load histories are inherently considered. On the other hand, the length of the monitoring period influences accuracy of the results. Therefore, long monitoring periods are preferred in order to being able to capture true distribution of load histories including rare traffic events. The monitoring period is divided into sections of 7-14 days and stress extremes are evaluated from each section. The extremes are then statistically evaluated.

Although it seems to be a promising approach in the use of monitoring data for capacity assessment, it has not yet been sufficiently tested and therefore in present time it is not recommended for general application.

Reliability of the bridge

Calculating reliability of the bridge is a much more ambitious approach. It is conducted in two steps:

- improving Finite-Element model using monitoring results (model updating)
- safety assessment of the bridge using the FE-model

Model updating is an inverse mathematical optimization procedure that modifies the FE-model in such way that its calculated response matches the measured bridge response. By this

way, the model is adjusted to reality. The procedure is computationally intense. Unfortunately, commercial software packages for solving this task are very rare.

Model updating can be performed using measured dynamic structural properties (eigenfrequencies and mode shapes). Most widespread is following definition of objective function:

$$J(p) = \Delta\varepsilon^T \cdot W \cdot \Delta\varepsilon + p^T \cdot W_p \cdot p \qquad \text{(Eq. 4.5)}$$

where $\Delta\varepsilon$ is the vector of differences between measured and calculated modal parameters,
p is the vector of structural parameters,
W, W_p are weighting matrices.
The measurement accuracy can be incorporated into the calculations by the weighting matrix

$$W = \left(\mathrm{cov}(\psi)\right)^{-1} \qquad \text{(Eq. 4.6)}$$

where ψ is vector of data extracted from measurements.

Alternatively, dynamic displacement residuals can be used as the objective function:

$$J(p) = \sum_{r=1}^{m} R_{G,r}{}^T \cdot R_{G,r} \quad , \qquad R_{G,r} = K^{-1} \cdot (K^t + i\omega_r^m . C^t - \omega_r^{m^2} . M^t) . \phi_r^m \qquad \text{(Eq. 4.7)}$$

where K^t, C^t, M^t are theoretical structural matrices (stiffness, damping, mass),
ω_r^m, ϕ_r^m are the r-th structural eigenfrequencies and mode shapes, respectively,
$R_{G,r}$ is r-th vector of dynamic displacement residual.

Model updating due its inverse character may in some cases react very sensitively to small variations of the inputs. To handle this problem of robustness, following points should be kept:

- the model must have few updating parameters
- expected structural damages produce unique structural response, which implies ability of distinguishing between particular damages
- the FE-model is a good representation of the real bridge, i.e. calculated structural response matches measured structural response in undamaged state

Measurement accuracy must be considered in the model updating procedure. Inaccurate data can produce false model updating results. The problem of data accuracy can be reduced using the weighting matrix.

Another approach is a procedure, which updates the model only if the changes of dynamic properties exceed a given confidence interval of the measurements (Figure 4.1). This way, the risk of false-positive damage identification is minimized.

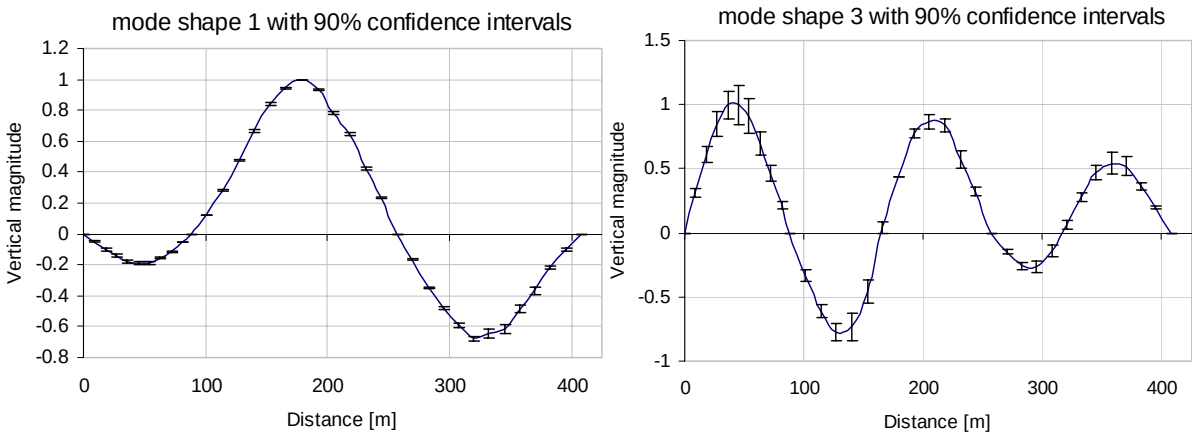


Figure 4.1 Example of measured mode shapes with 90% confidence intervals

The model updating procedure was tested on a concrete bridge (chapter B3.1.3, Annex B). Measurements of the bridge were available in its undamaged state. The damaged states were simulated as stiffness decrease in selected regions. The damage identification results showed a non-unique solution, where several damage locations were identified.

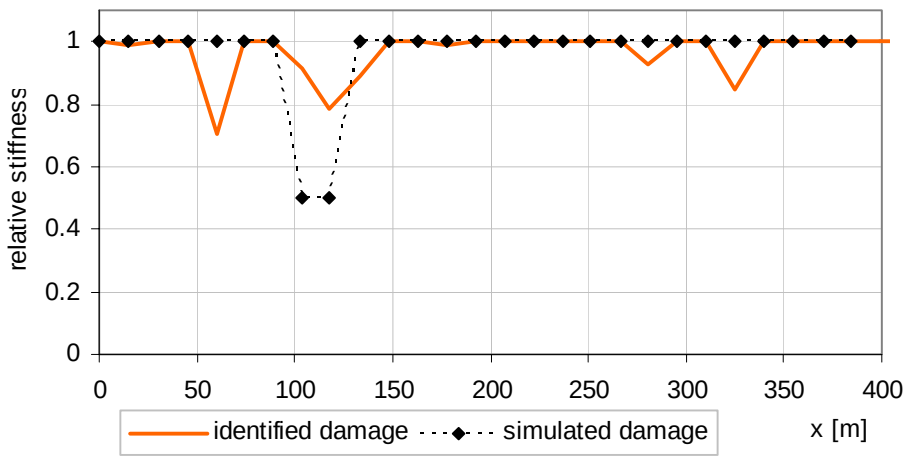


Figure 4.2 Damage identification example

This example shows that results of model updating cannot be treated as an exact solution. The provided result gives hints, which damage locations could lead to the recorded structural response. In the case of positive damage identification, the analyst must perform a plausibility check, which may include a visual inspection. As a result, some model parameters can be removed from the model, which improves the solution accuracy.

If an updated FE-model has been acquired, it can be used for computational safety assessment.

The safety assessment can be done by reliability analysis, where the failure state of a system is represented by a function of the response known as the limit state. Reliability analysis consists of estimating the probability of failure; that is, the probability that demand will exceed the system's capacity. The probability of failure P_f is defined as the integral of the Joint Probability Density Function (JPDF) of demand *S* and capacity *R* over the failure or unsafe region (Figure 4.3).

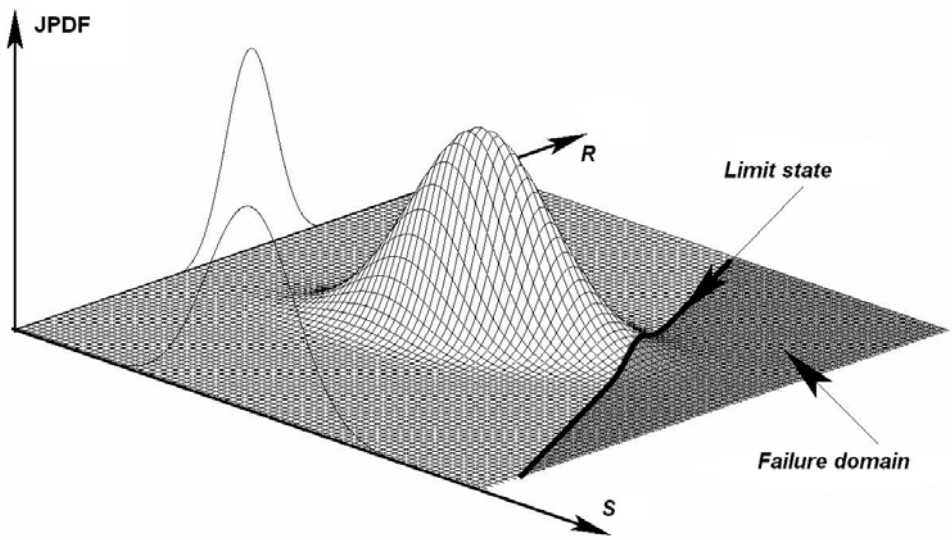


Figure 4.3 Joint probability density function of strength (capacity) X1 and load (demand) X2

The given task of calculating P_f is computationally very demanding, therefore few approximations must be met:

- numerical integration of P_f through statistical sampling (Latin Hypercube Sampling)
- Taylor series expansions of the limit state function at a design point (closest point of the limit state function to the origin)

Assessing reliability of the bridge using model updating based on monitoring data is a very ambitious approach.

The advantages are:

- the whole structure is being assessed
- failure modes are more precisely described, since force redistribution after failure of structural components can be considered

Disadvantages are:

- high data needs require an extensive monitoring system with many sensors

- very high computational effort in both model updating and reliability index calculation, both methods are not yet widespread in commercial software packages
- model updating can deliver non-unique solutions, which may largely affect reliability index calculation

Due to its disadvantages, the application of this approach is recommended only in very limited cases.

Short-term monitoring

Results of short-term monitoring can be used to confirm certain design assumptions. Recommended application is the measurement of stress in external tendons (e.g. by the magnetoelastic method).

Short-term monitoring can provide more interesting results if it is applied as measurement system during a **load test**. This application is described in more detail in Deliverable D16.

4.3 Extension of service life of deteriorating bridges

The deterioration of bridges is assessed during regular visual inspections. The assessing engineer has the task to find visible signs of deterioration and to assess their influence on the structure.

Monitoring of a bridge that shows signs of deterioration is recommended, if there is high risk that deterioration would reach unacceptable levels until the next visual inspection. More frequent visual inspections may also be a solution in some cases. If the repair of the detected deterioration problem is impossible, not economical, or otherwise currently not desired, monitoring system can help to extend the usable lifetime of the bridge or to postpone repair works by means of monitoring of deterioration propagation.

There is a number of potential structural deficiencies that can be monitored:
- Crack width
- Strain level in critical components: strain reserve to yielding or post-yielding
- Stress in external cables: stress reserve to allowable stress, fatigue accumulation
- Displacements: excessive deformation (serviceability)
- Vibration: excessive vibration levels (serviceability)
- Expansion joints: permitted relative movement
- etc.

Design of the monitoring system is straightforward, since the location of the structural defects is known. Structural analysis must be carried out in order to determine limit values for the monitored quantities. The measurement evaluation is then simple, since it consisting of only testing if the monitored parameter does not exceed a given level.

In cases where absolute values of monitored quantities cannot be directly measured (like stress in concrete), their value at beginning of monitoring (X_0) has to be estimated and its relative change (ΔX) is measured.

$$X_0 + \Delta X + X_{\text{var}} \leq X_L \ , \qquad \text{(Eq. 4.8)}$$

, where X_0 is estimated parameter value at beginning of monitoring,
ΔX is measured change of the parameter (normalized),
X_{var} is estimated parameter change due to traffic loads,
X_L is limit value of the parameter.

Monitoring can assist in maintenance of deteriorating bridges. Two possible profits are:

- rehabilitation of the bridge can be postponed until a desired level of monitored parameters is reached.
- bridges towards end of their lifetime can be used longer

4.4 Extension of service life of bridges of special importance

Monitoring can be applied on structures without known problems as a preventive measure. The task of such monitoring system is early detection of structural problems. Due to low probability of occurrence of structural problems, questions may arise about profitability of the system. Generally, preventive monitoring would be considered profitable for bridges that would produce very high losses in case of failure, functional interruption or late maintenance measures – which is summarized in the term “bridges of special importance”.

Monitoring system design has to be carried out very carefully. Risk analysis and sensitivity analysis are two major tools in this task. Risk analysis identifies probable structural problems. Sensitivity analysis calculates structural response in case of occurrence of an assumed damage and tries to identify parameters of structural response that are most sensitive to the given type of damage. As a result, optimal monitoring system layout can be designed.

Structural system changes calculated by the sensitivity analysis have to be compared to the accuracy of the available sensors. High ratio of expected change / sensor accuracy is a good premise for damage identification ability.

Sensitivity analysis has been performed on an example of a prestressed concrete bridge. The considered damage scenario was failure of prestressing tendons above a pier. Most sensitive parameters were:

- strain
- inclination
- eigenfrequencies

Optimal positions (most sensitive) of the sensors result from the sensitivity analysis (chapter B2.1.4, Annex B).

Measured data should be normalized (chapter 4.5) to minimize environmental influences.

Evaluation of the measurements is carried out by assessment of a damage indicator (chapter B3.1, Annex B). The desired level of damage identification is level 1 – detection of presence of damage. Identification of damage location and severity are not required in order to avoid complexity of the system.

Data mining techniques are recommended for processing of measured data. In chapter B3.1.2, Annex B, an application of clustering techniques is shown. The presented Sens Rail Bridge in France was tested before, during and after strengthening. Clustering techniques were used to

match the measurements with particular structural conditions. Best results were achieved by the hierarchy-divisive method, where measurement before and after strengthening could be correctly identified.

Accuracy of the data extracted from measurements must be considered in damage identification. The accuracy can be evaluated from measured data by statistical evaluation.

When accuracy is evaluated, the ability of damage detection can be proved. The structural changes due to damages defined by risk analysis should be identified despite limited accuracy of measurement data. An example of damage identification ability from structural eigenfrequencies is given in chapter B3.2, Annex B. A prestressed concrete box-girder bridge was analysed. Selected damages (concrete cracking) were modelled and their influence on eigenfrequencies was calculated.

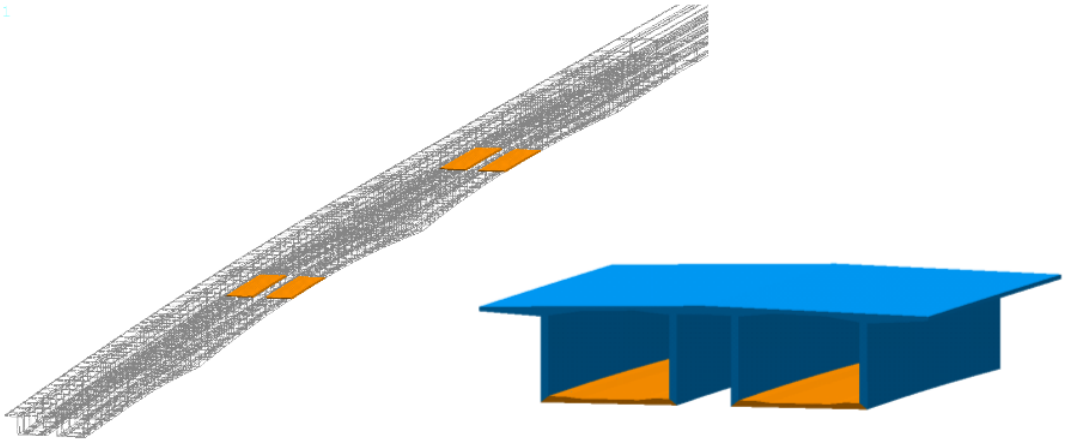


Figure 4.4 Selected areas of reduced concrete stiffness

The expected frequency changes were low (below 1%). The result of the analysis showed that small damages cannot be identified. The damage is identifiable if it produces a frequency change of ca. 1 standard deviation in each eingenfrequency.

Table 4.1 Identification ability of simulated damages

E_c reduction	$\Delta f_1 / \sigma(\Delta f_1)$	$\Delta f_2 / \sigma(\Delta f_2)$	$\Delta f_3 / \sigma(\Delta f_3)$	Identifiable?
10 %	-0.3	-0.5	-0.2	No
25 %	-0.7	-1.4	-0.5	Relatively well
50 %	-1.6	-3.1	-1.2	Yes

Primary task of preventive monitoring systems is early damage detection. However, the monitoring system can also feature elements for reliability assessment of structural components, described in 4.2.

If damage was identified during monitoring, the purpose of the monitoring system is fulfilled and further analyses need to be done. This may include visual inspection and also extension and redesigning of the monitoring system.

4.5 General system recommendations

The principles listed in this chapter are valid for all monitoring systems.

The basic architecture of monitoring systems is given in chapter B2.2, Annex B. The sensors should be chosen according to their application. Following criteria should be considered during sensor selection:

- accuracy
- measurement range
- temperature sensitivity, cross-sensitivity
- long-term stability of the signal
- weather sealing, connectivity, etc.

Long-term stability is important for static measurements and for capturing slow changes. Foil-type strain gauges are recommended only for dynamic measurements because of their slow signal drift.

If the influence of temperature is an undesired component in the results, the measurement data must be normalized (Sohn, 2007) in order to compensate temperature changes (chapter B3.2, Annex B). Most common techniques for data normalization are autoregressive models with exogenous inputs (ARX) or nonlinear regression analysis. The latter was applied for evaluation of structural eigenfrequencies of a prestressed concrete bridge. The data normalization is used to reduce the uncertainty of the results (Figure 4.5).

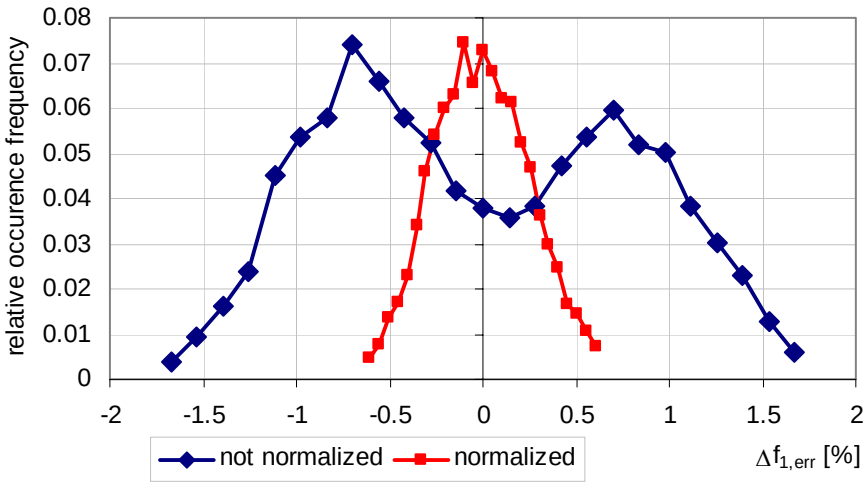


Figure 4.5 Deviations of first eigenfrequency

5. ACOUSTIC EMISSION (AE)

This chapter summarizes the results of work on Acoustic Emission technique. Acoustic Emission technique is an emerging monitoring technique that has gained attention because its potential is not yet fully known. Thematically it belongs under the topic monitoring-based assessment next to other monitoring techniques, but here the AE was investigated more extensively and therefore is presented in this separate chapter.

More detailed information on Acoustic Emission is included in Annex C.

5.1 Principle of Acoustic Emission technique

Acoustic Emission can be used as a part of monitoring system during:

- long-term monitoring
- load tests (short-term measurements)

In this recommendation we will concentrate on the second type of usage in the context of bridge load testing and bridge evaluation.

Acoustic emission monitoring of bridges requires application of mechanical or thermal stimulus. Test procedure for using of AE data acquisition systems during soft, diagnostic and proof load testing of bridges will be described in this recommendation.

AE works by detecting transient elastic waves generated by the release of energy within a material. The release of energy may be caused by different sources.

Basically there are four categories of sources:

- Primary AE are generated when new damage/disintegration occurs, i.e. by an overload event where the maximum previous stress level is approached or exceeded.
- Secondary AE are generated where the maximum stress level does not exceed the previous threshold.
- Undesired noise is transmitted through the test setup (between the specimen/structure and the force application plates/vehicles).
- Artificial Sources are commonly used for calibration purposes or to study wave propagation.

Reinforced concrete is a composite material and has a larger number of source mechanisms than steel or plastic. Overview of possible AE sources in reinforced concrete is mentioned in Table 5.1.

Table 5.1 Overview of possible AE sources in reinforced concrete (ODOT, 2008)

Effect	Cause/description	Category
Micro crack generation	Shrinkage, temperature, creep, low load effects	Primary, distributed
Macro crack formation and	Load effect due to shear, moment, or	Primary,

propagation	tension forces	from crack tip
Concrete crushing (plastic deformation)	Concrete in compression zone	Primary
Steel rebar yielding/fracture (plastic deformation)	Steel in tension, overload event, low-cycle fatigue	Primary
Rebar de-bonding (at crack planes, after crack formation)	Repeated differential loads (i.e. live loads)	Primary
Crack surface rubbing, interaction between steel rebars/ concrete	Repeated differential loads (i.e. live loads)	Secondary
Artificially generated signals	Sensor pulse/ pencil break (Hsu Shoe)	Calibration, surface
AE generated from outside the body	Experiment: slip/friction in test frame and bearings Field: tire friction, uneven surface causes vehicle bouncing, studded tires	Undesired noise, surface
Artificial AE from within the electrical circuit/AE system	Power supplies, cables, cell phones	Undesired noise, electr.

5.2 Measurement system

AE measuring system shall consist of the following devices:

- AE sensor,
- pre-amplifier and amplifier,
- A/D converter,
- computer.

AE sensors shall be sensitive enough to detect AE signals generated in the target structure, taking acoustic coupling into consideration. The AE sensors used are usually resonance sensors, which are only very sensitive to a certain frequency, while a broad-band sensor has approximately flat response in the range but is less sensitive than the resonance-type. Resonant type sensors (60 kHz to 150 kHz) are preferred to be used due to greater sensitivity and potential coverage then higher frequency units. High-fidelity sensors are generally not sensitive enough to be of practical use for field testing.

Proper sensor location is vital for successful AE based evaluation of the structure.

The array deployment should be decided considering the sensor type, number of sensors and desired area of coverage. For general application, a widely spaced linear array deployment will provide the greatest coverage. Sensor spacing up to approximately 200 cm can usually be used with the 60 kHz sensors and just 50 cm for 150 kHz sensors, but shall be individually

defined. Depending on the number of sensors available more then one linear array may be desirable.

If there is one particular damage feature that is of concern then using planar arrays that cover the defect area are preferable. Ideally enough sensors can be employed to deploy two or more planar arrays of 3 channels or more around the defect area. When determining the exact location of each sensor, it should be kept in mind that non-symmetric sensor placement around the defect will provide the greatest accuracy for localization algorithms if they are to be used. Mounting sensors on both sides of the stem and the bottom face is desirable for optimal coverage of the damaged region. It is also desirable to locate and mark of all stirrups in the test area using a rebar locator.

The AE system can tolerate lead wire runs exceeding 60 m if necessary, but those of other sensor types like crack moth opening displacement (CMOD) and particularly strain transducers will not, unless signal conditioning can be applied at the transducer. If a medium to long term health monitoring system is to be employed, then vandalism and theft of the expensive test equipment must also be considered.

The parametric sensors should be installed prior to mounting the AE sensors, especially if strain gages are to be used. This will reduce the chances of damaging the AE sensors.

With the data acquisition system up and running the triggering thresholds for each AE channel should be set. Ideally five minutes of data with no alternating loads on the structure should be measured. The RMS levels from each sensor can be used to determine the lowest threshold levels which for a quality AE system and sensor should be around 3 times the RMS value. Floating threshold levels can be implemented with good success on AE systems so equipped. Once the thresholds are set ambient traffic should be allowed to cross the structure. The recording threshold, which is typically higher then the detection threshold, can be set by observing the response of the system to insignificant loads such as small passenger cars.

Once the data acquisition system is fully operational and threshold set, the controlled loading can be conducted. It is almost mandatory that no other alternating and preferable no additional static load are on the test span or the attached approach spans during the application of the controlled loads. With highway bridges this can be challenging and will require proper planning of traffic control. On high volume highways the rolling blockage performed in low volume hours of operation is recommended. Typically under these conditions static load cases cannot be applied. If permitted, static load cases are desirable for unambiguous calculations of the Calm Ratio.

The AE can be applied during soft load, diagnostic or proof load tests. More information about particular test types can be found in Deliverable D16.

5.3 Evaluation

This chapter summarizes important parameters that can be used in evaluation of AE measurements. More detailed description can be found in chapter C3, Annex C.

There are two types of emissions or AE signals which can arise during application of stimulus on a structure: burst emissions or signals and continuous emissions or signals.

The burst emission means that acoustic emission events can be separated in time. This type is emitted in the case of crack initiation or propagation in concrete, for example.

The continuous type emissions result from continuous events such as leaking.

Dealing with AE based monitoring or evaluation of concrete structure the burst emission signals are of interest.

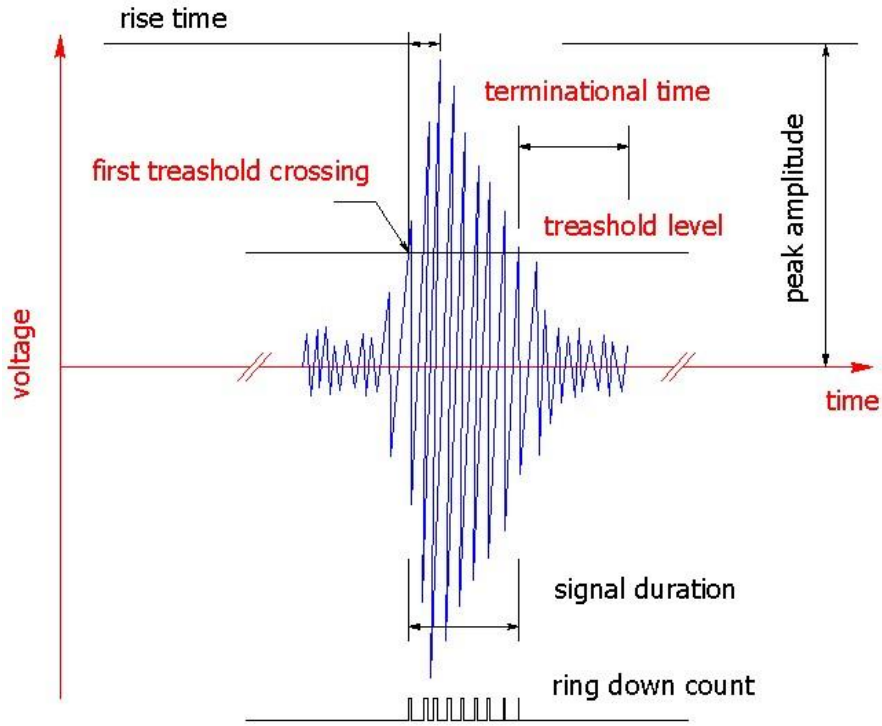


Figure 5.1 Four basic parameters of a burst signal according to EN 1330-9

The AE parameters which shall be recorded depend on the goal of the investigation. Usually the following AE parameters should be obtained by the measuring system:

- density of hits,
- duration of the signal,
- rising time of the signal,
- number of counts,
- amplitude of the signal and
- absolute energy.

In addition to these parameters when source location is of interest the arrival time differences in AE sensor array shall be obtained by the measurement system too.

The measurement system used shall be able to obtain time information along with AE parameters and external parameters such as load, strain and other relevant data.

AE test results can be used for the detection and monitoring of damage evaluation, damage location, determination of damage level, for the determination of the crack type (tensile, share, mix) and for the determination of damage source.

Basically speaking the acoustic activity is caused by damage occurrence. Stress level at which the AE events occurred is important. The lower the stress at which the AE activity starts, the poorer is the structure.

Total number of AE events is significant. The larger the AE event rate, the grater the damage to the structure.

The AE can be used as a failure warning. Large increases in AE amplitudes normally indicate that the structure is near to the failure level. The higher the energy (amplitude) of an AE event, the larger is damage to the structure.

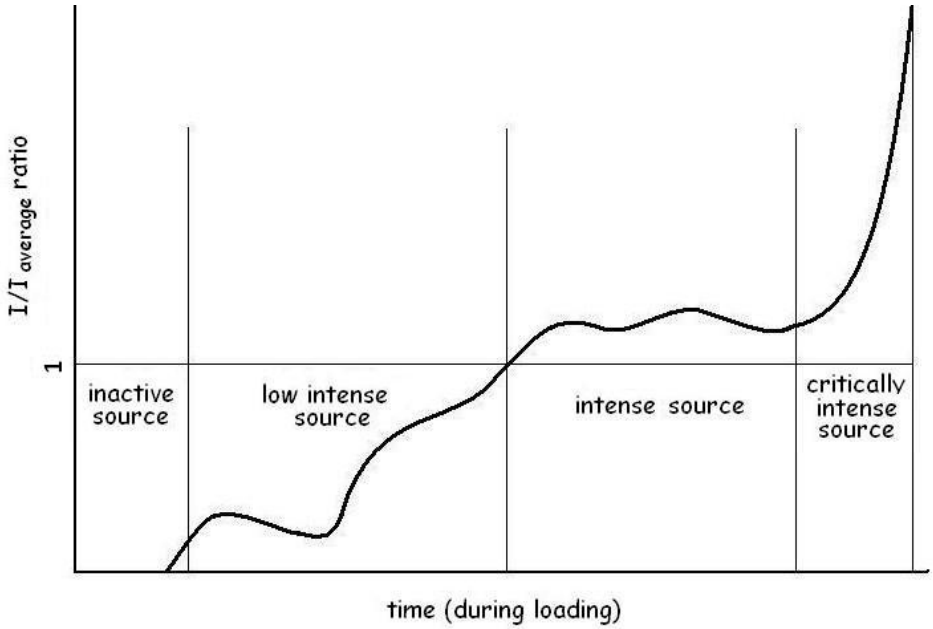


Figure 5.2 Different types of source with respect to its AE intensity (according to ASTM E569)

The rate of accumulation of AE events as a function of increasing stress (time) is significant. When the slope of such a curve changes significantly, the rapid growth of damage indicates changes in source mechanism or flow growth becoming unstable as a precursor to total failure.

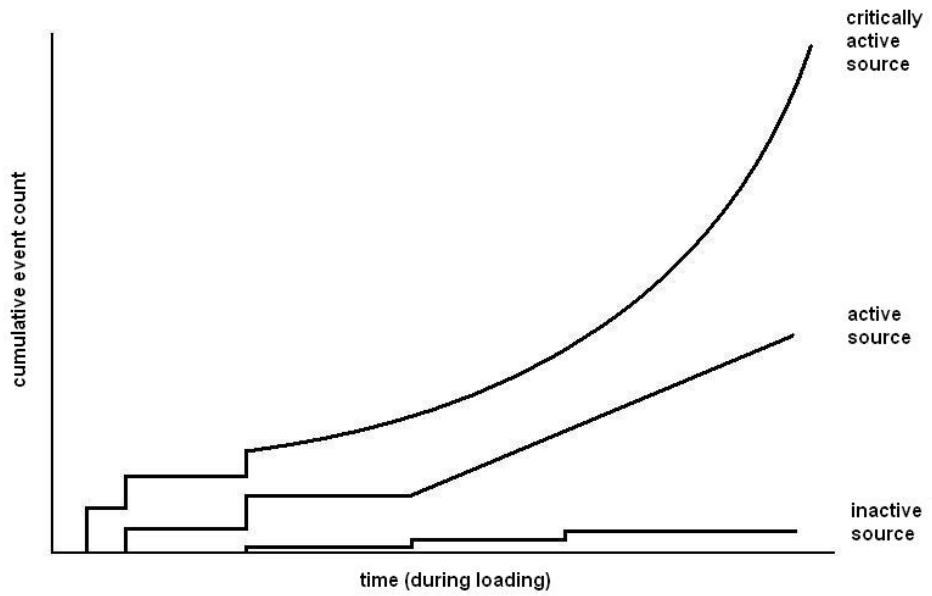


Figure 5.3 Different types of source with respect to its AE activity (according to ASTM E569)

Location of the AE sources within structure is of key significance. Of much greater importance are AE events which originate at the same location. These AE events indicate a growing region of damage and a potential serious damage to the structure.

Kaiser effect is called 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.

Appearance of significant acoustic emission at a load level below the previous maximum applied level is called Felicity effect. Felicity effect was observed in composite materials like concrete.

The level of the AE activity during the unloading can be used to evaluate the damage level of the structure. The higher the AE activity during the unloading, the greater is the damage level. Value of the Felicity ratio is a significant factor. The lower the value of the Felicity ratio, the poorer is the structure/specimen.

$$\text{Felicity Ratio} = \frac{\text{Load at which significant emission restarts}}{\text{Previously applied maximum load}}$$

A decreasing Felicity ratio corresponds to a growing damage in the structure. Felicity Ratio value greater than 1 implies that the Kaiser effect still holds for the structure.

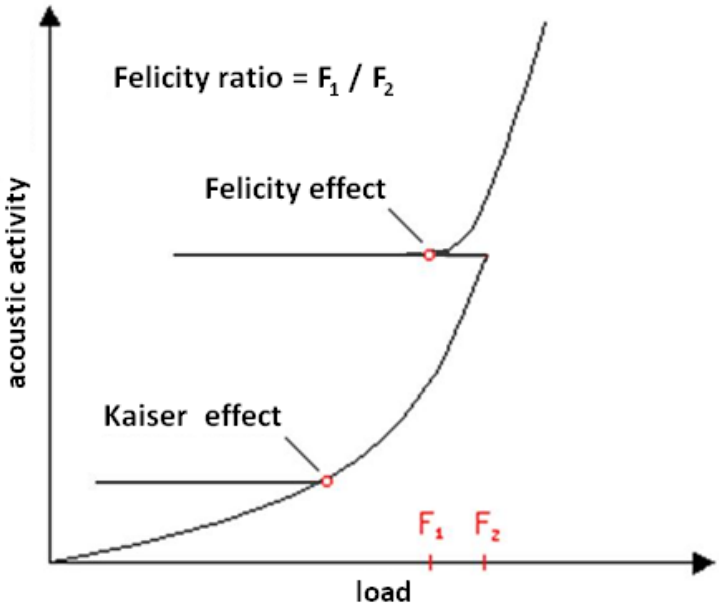


Figure 5.4 Kaiser effect and Felicity effect

Damage quantification can also be more precisely evaluated based on the Calm and Load ratio. The number of AE events during unloading divided by the number of events during the whole loading and unloading cycle is defined as the Calm ratio. Values near zero indicate intact material condition. Load ratio is defined as stress at which AE activity starts to generate divided by the maximum stress. A complete unloading of the structure is needed in the case of Calm and Load ratio analysis. Using the Calm and Load ratio, the NDIS diagram (Figure 5.5) can be obtained and the level of damage deduced.

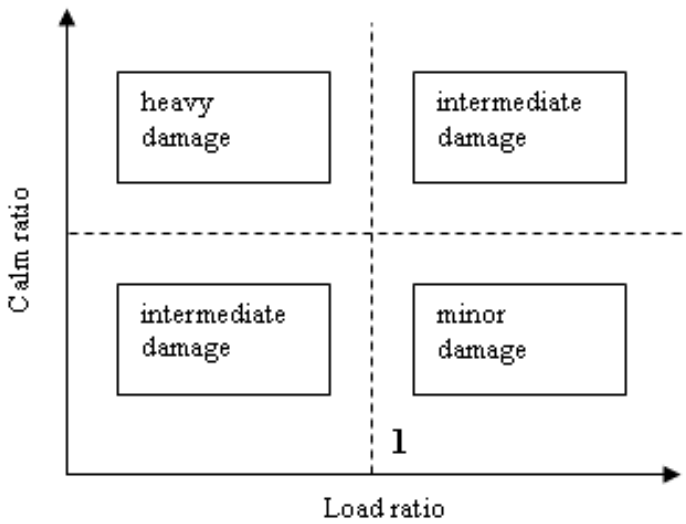


Figure 5.5 NDIS diagram

The qualification of the damage (tensile crack vs. other-type crack) can be performed based on the AE waveform parameters.

Intensity analysis is a measure of the structural significance of an AE source. It consists of two parameters: the Historic Index and the Severity. The Historic Index compares the signal strength of the most recent hits with the signal strength of all hits up to that point. The Severity index computes the average of the largest signal strengths. The severity of damage increases when both indices increase (see chapter C3.6)

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. *b*-value analysis appears especially well suited for implementation in a structural health monitoring system since it is computationally inexpensive and, theoretically, only one sensor is needed (see chapter C3.7).

Relaxation ratio is ratio of average energy during unloading to the average energy during loading phase, where the average energy is calculated as the cumulative energy recorded for each phase of loading divided by the number of recorded hits. A relaxation ratio grater than one implies that the average energy recorded during the unloading cycle is higher than the average energy recorded during the corresponding loading cycle, therefore the relaxation (aftershock) is dominant. Vice-versa the loading (foreshock) is dominant (see chapter C3.8).

Moment tensor analysis allows location of acoustic emission sources and distinction of different types of cracks generated during loading of concrete samples/structures. Crack location, crack type (shear, tensile and mixed) and orientation can be quantitatively identified by SiGMA (simplified Green's functions for moment tensor analysis) procedure (see chapter C3.9).

Frequency analysis - frequency spectrum is calculated from the time domain of AE signal. In a lot of cases frequency spectrum of recorded signals provides detailed information about AE sources which cannot be obtained by parametric analysis of the time domain of AE signal such as steel breaking, steel corrosion, concrete cracking, steel-concrete interface damaging etc (see chapter C3.10).

5.4 Application examples

The evaluation of AE measurements was tested within the ARCHES project in several laboratory tests and two "in situ" tests.

5.4.1 Test at ZAG with reinforced concrete specimens

The tested specimens were prisms with dimensions 10x15x80 cm. They were loaded in bending and the cracking process has been analysed.

Cyclic and non-cyclic loading were performed. Cycling loading was performed in order to analyse the presence and the use of the Kaiser Effect.

Damage location was also identified.

The first crack in the concrete appeared when hits density reached its peak. In following load cycles, recorded hits density was lower and cracking was well. After reaching the phase of plastic behaviour, AE activity increases again, most probably as a result of the pull-out of the rebar from the concrete.

The AE activity clearly followed the uploading and unloading cycles. During the unloading the AE activity died out, and appeared again when the load level of the previous loading cycle was reached. The presence of the Kaiser effect could be proved.

Identification of damage locations showed good results, corresponding well with the observed crack pattern. In the presented example, two major cracks were observed (Figure 5.6).

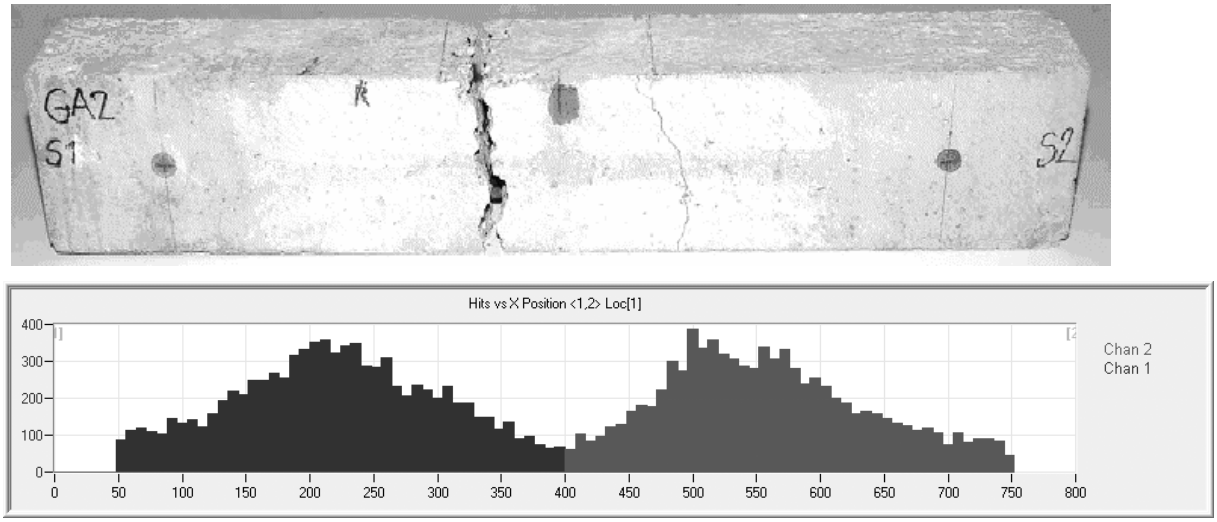


Figure 5.6 Hits vs. X position (mm) for specimen GA2

Detailed test results can be found in Chapter C4.1, Annex C.

5.4.2 Test at ZAG with old bridge girders

The tested bridge girder was taken out from a 40-year old reinforced concrete bridge with a span of ca.11.5 m. Cycling load test was performed using four loading points.

The AE activity increased as the applied load approached the load bearing capacity of the girder – the highest AE activity was recorded at the peak load.

During each unloading cycle the AE activity died out and reappeared again during the loading phase, before the load level of the previous cycle has been reached. This means that the Kaiser effect could not be proved on this girder, but the Felicity effect was observed.

Damage severity was evaluated by the Calm and Load ratio, and the qualification of the damage (tensile crack vs. other-type crack) was based on the AE waveform parameters.

The recorded Calm and Load ratio corresponded well with the NDIS diagram. Figure 5.7 shows the test results, the NDIS was inserted into background of the figure.

Detailed test results can be found in Chapter C4.2, Annex C.

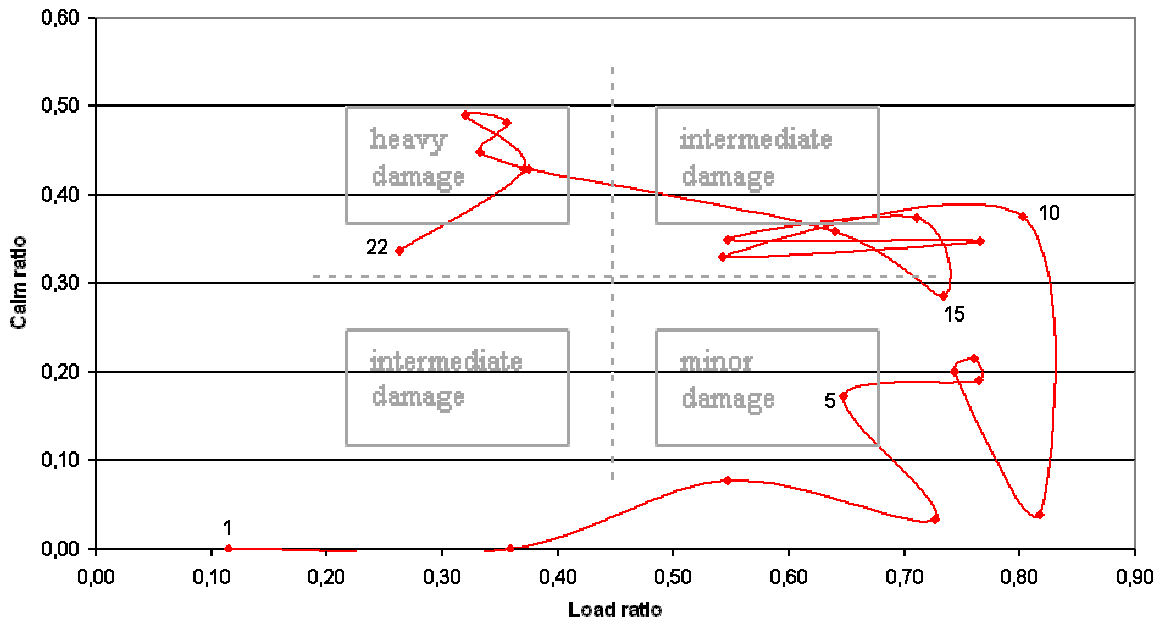


Figure 5.7 Calm and Load ratio (the numbers near the markers indicate the loading cycles)

5.4.3 Barcza Bridge Test

During the Barcza bridge test the proof load test was supported by the Acoustic Emission method. The AE results were compared with the visual inspection and the load-deflection and load-strain diagrams.

It was shown that based on the AE signals it 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. 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. Figure 5.8 shows the measured strain-load diagram, where the red vertical line marks the loading level where the cracking was detected by visual inspection and 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.

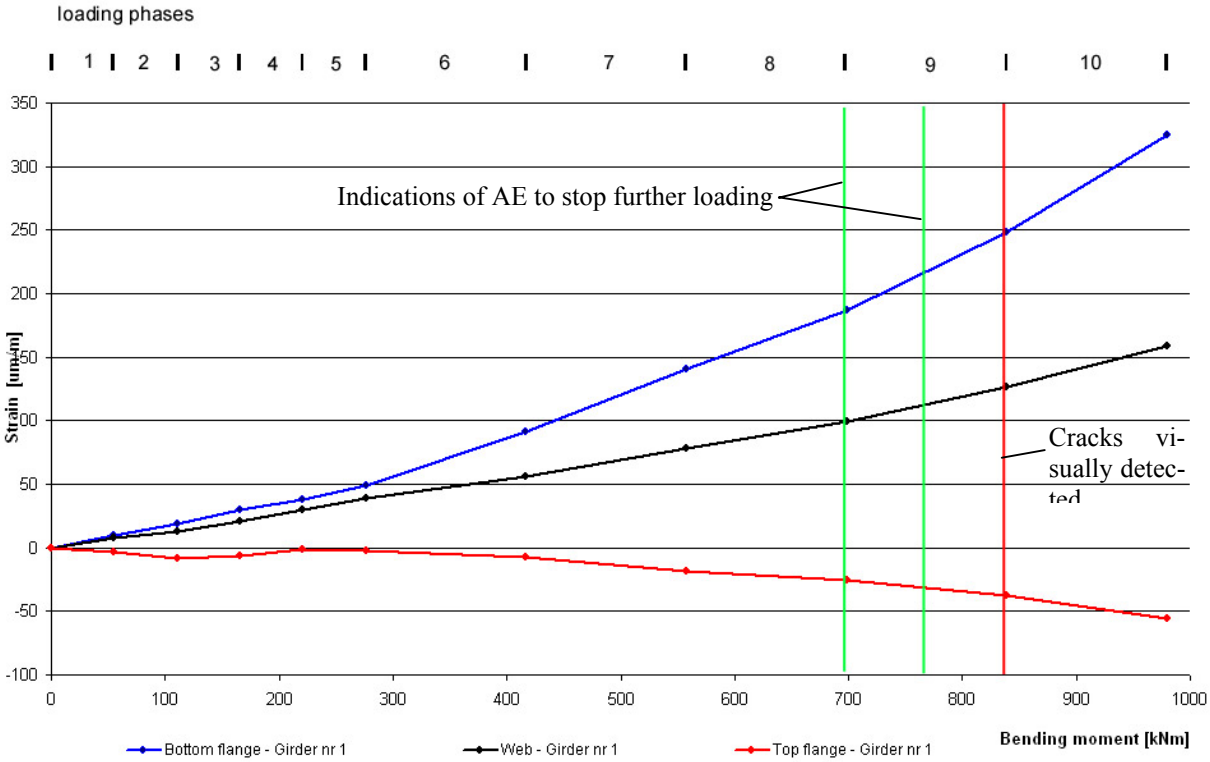


Figure 5.8 Strain-bending moment diagram in girder no.1

Detailed test results can be found in Chapter C4.3, Annex C.

5.4.4 AE frequency analysis on Prague Lahovice bridge

The second on-site test was carried out at the Prague Lahovice bridge. The main idea of the test was to study possibilities to identify corrosion of rebars by AE method.

During static or pseudo-static loading test of bridges the load level necessary for emission of acoustic signals was difficult to be reached. Therefore an impact loading provoked by a loaded lorry driven over an obstacle was used.

Based on the interpretation of time domain of recorded signals during dynamic loading the integrity of individual bridge girders can be evaluated.

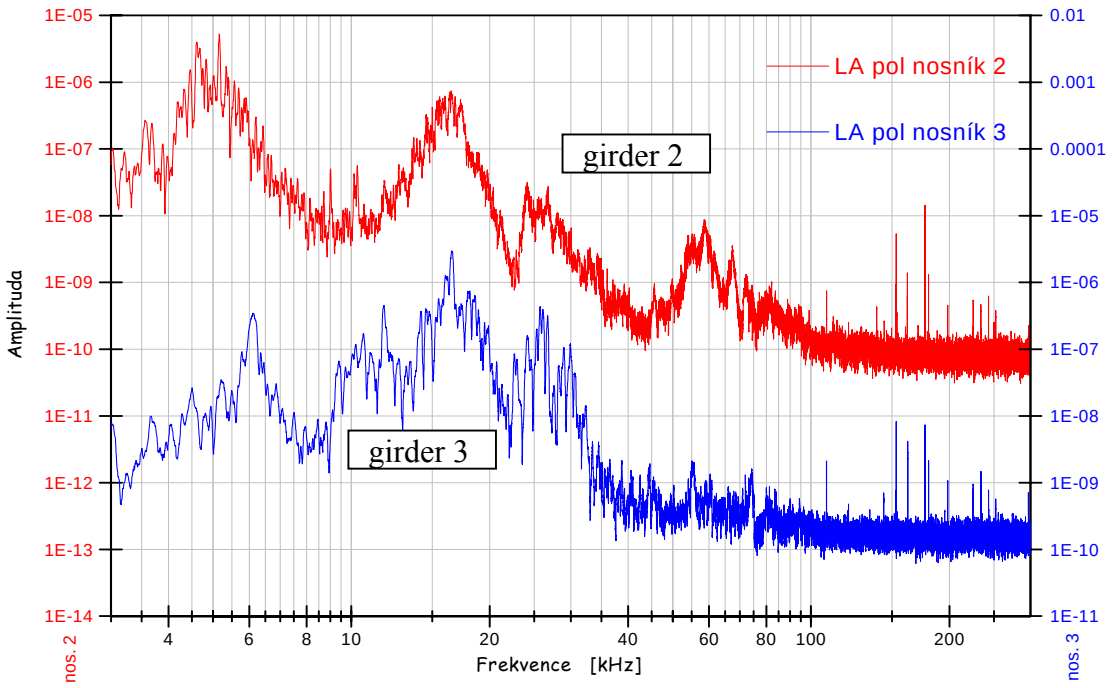


Figure 5.9 Frequency spectra – inner girders 2 and 3 measured in the middle of the bridge span

Detailed test results can be found in Chapter C4.4, Annex C.

5.5 Short conclusion summary

The conclusions that could be gained as result of the performed tests are summarized in the following points.

AE is a very perspective non-destructive method for the detection and localization of active cracks in reinforced concrete specimens.

- The acoustic activity is highest at formation of the first crack, probably when the tensile strength of the concrete is reached.
- The event rate is the most convenient parameter for the detection and monitoring of acoustic emission activities.
- NDIS criteria can be used to assess the extent of damage.
- Intensity charts generated from Historic index and Severity values are also a good indicator of cracking activities. The Historic Index compares the signal strength of the most recent hits with the signal strength of all hits up to that point. The Severity index computes the average of the largest signal strengths. The severity of damage increases when both indices increase.
- Felicity ratio can be used as indicator of structural condition. The higher the AE activity during the unloading, the greater is the damage level.

- In the unloading stage, the AE activity is highly reduced and even dies out completely. According to Kaiser Effect, the AE activity should reappear when the maximum load of the previous cycles is reached. The presence of Kaiser effect was proved at some specimens, while other tests did not show this effect
- AE can be used for the detection of source location

AE technique is a very promising non-destructive method for the detection of active cracks in reinforced concrete elements.

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Figures

Tables