Structural assessment of bridges using monitoring data

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This dissertation is submitted for the degree of
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To my family, mentors and friends
Declaration

I hereby declare that except where specific reference is made to the work of others, the contents of this dissertation are original and have not been submitted in whole or in part for consideration for any other degree or qualification in this, or any other university. This dissertation is my own work and contains nothing which is the outcome of work done in collaboration with others, except as specified in the text and Acknowledgements. This dissertation contains fewer than 65,000 words including appendices, bibliography, footnotes, tables and equations and has fewer than 150 figures.

Cong Ye
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Abstract

Despite decades of research, there has been relatively little industry uptake of structural health monitoring (SHM) technologies and model updating techniques, i.e., the integration of structural monitoring data with structural analysis modelling, in bridge operation and maintenance (O&M) activities. Many bridge practitioners question whether these technologies and techniques can provide valuable and reliable information on bridge safety and performance.

In the first part of this PhD thesis, a series of industry interviews are used to investigate the reasons behind this limited industry adoption and the disconnects between research and practice regarding bridge monitoring and model updating. It has been found that while most studies in the bridge SHM community have been focused on damage detection, many bridge practitioners are ultimately more interested in bridge capacity assessment, and in particular, the “margin of capacity” of their bridge assets (i.e., how much additional live load can be safely placed on a bridge). In addition, bridge assessment remains a key asset management challenge in that there has been a consistent mismatch between the calculated load rating and the actual capacity of bridges.

These findings form the basis of the second part of this PhD thesis, which investigates how strain monitoring data may be utilised to improve bridge assessment by (i) better understanding and quantifying commonly made assumptions about structural behaviour to enable more realistic bridge modelling and analysis, and (ii) evaluating in-service structural utilisation to better understand the “margin of capacity”, which is defined in this PhD study as how much additional live load can be safely placed on a bridge without violating the design performance criteria.

First, four common assumptions about structural behaviour in bridge assessment are examined. These are the amount of prestress loss (for prestressed concrete bridges), load distribution characteristics, support boundary conditions and the stiffness contribution of secondary structural elements. Novel or refined methods are developed to enable the evaluation of these structural behaviour characteristics under normal operational conditions. In particular, the methods developed utilise normalised structural response profiles to minimise the effects of uncertainties in estimating material stiffness and load magnitudes. Using these
methods, continuous data collection for more automated and reliable evaluation of in-service structural behaviour is made possible.

Subsequently, to improve the understanding of “margin of capacity”, a new and systematic methodology for evaluating and visualising in-service structural utilisation of bridges, based on monitoring data, is presented. Three definitions of structural utilisation and three types of visualisation are proposed to inform the “margin of capacity”. It has been found that the actual utilisation, based on monitoring data, can be significantly lower than the design expectation. Finally, a comprehensive analysis of data-related uncertainties is presented in order to evaluate the uncertainty levels of the output parameters of interest (e.g., structural properties, load effects) in the aforementioned studies. This study also includes a sensitivity analysis to identify the sources of data-related uncertainty that are most significant.

Building on the outcomes of this thesis, it is envisaged that the industry challenge of “How much additional live load can be placed on a bridge before violating its safety, serviceability or durability criteria?” could be addressed, thereby facilitating more efficient utilisation and more targeted maintenance of bridge assets and hence realising the practical value of bridge monitoring and model updating.
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Chapter 1

Introduction

1.1 Societal and industrial context: bridge management and maintenance

*The life of a metal bridge that is scientifically designed, honestly and carefully built, and not seriously overloaded, if properly maintained, is indefinitely long.*

— John A. L. Waddell (1921)

Managing and maintaining existing civil infrastructure assets, such as roads and bridges, is of ever-increasing importance. This is due to two growing challenges at the present time and going into the future:

1. **“Asset time bomb”**: A large number of existing assets are approaching their theoretical end-of-life state at the same time (Thurlby, 2013), as they are subjected to increasing frequency and magnitude of load and exhibit significant levels of deterioration.

2. **Climate emergency**: The remaining carbon emission budget for reaching a 1.5°C or 2°C warming scenario is depleting rapidly. As of 2021, based on the estimated global carbon emission rate, there are only approximately 6 years and 24 years left for 1.5°C and 2°C scenarios, respectively (Mercator Research Institute on Global Commons and Climate Change [MCC], 2021).

Bridges, in particular, are critical components of infrastructure systems. They act as points of interdependency in transportation networks and their performance is critical to the resilience of our urban environment.
The “asset time bomb” problem is a pressing issue for bridge infrastructure in many parts of the world. For example, in the U.K., as of 2018, 3,177 council-maintained road bridges were rated as sub-standard and the budget for necessary repair works has been limited (RAC Foundation, 2019). In the U.S., as of 2019, 47,000 out of its 616,000 bridges (21%) were rated as structurally deficient and the pace of repairs of these bridges has been slow (American Road & Transportation Builders Association, 2019). Both the Bridge Owners Forum (BOF) in the U.K. and the American Association of State Highway and Transportation Officials (AASHTO) have identified bridge operation and maintenance (O&M) related issues as the top of their grand challenges in bridge engineering and management, as shown in Table 1.1 (Bridge Owners Forum [BOF], 2020; Mertz, 2013).

Table 1.1 Top grand challenges of bridge engineering and management identified by BOF and AASHTO.

<table>
<thead>
<tr>
<th>No.</th>
<th>Grand challenge Identified by BOF</th>
<th>Grand challenge Identified by AASHTO</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Prevent bridge failures</td>
<td>Extend bridge service life</td>
</tr>
<tr>
<td>2</td>
<td>Extend the life of existing structures</td>
<td>Assess bridge condition</td>
</tr>
</tbody>
</table>

In addition, the built environment accounts for a significant proportion of global carbon emissions. For example, in the U.K., the civil infrastructure sector accounted for approximately 69% of all carbon emissions in 2018, of which 33% came from the transport sector (e.g., roads and bridges) (Goodwin, 2019). Minimising the amount of new build, for example, by improving the maintenance and utilisation of existing bridge assets, can have a significant impact on reducing carbon emissions to tackle climate change (Enzer et al., 2013; Norman et al., 2020). Specifically, this includes extending the remaining service life of bridge assets by more targeted and effective maintenance actions, utilising the true reserve load capacities more efficiently by adopting more realistic legal weight limits, and reducing traffic demand by optimising the combination of load and route (Allwood and Cullen, 2012; Greening et al., 2019).

More realistic assessment of structural capacity and structural utilisation for bridges is critical to facilitating more targeted maintenance and more efficient utilisation of bridge assets. Despite large increases in traffic loading over the years, many bridges have been able to remain in service (Hayward, 2011). Many studies have shown that there has been a mismatch between assessment ratings and actual capacities of bridges (Bakht and Jaeger, 1990; McConnell et al., 2015; Puurula et al., 2015). This is due to many potentially conservative assumptions commonly made in bridge modelling and analysis, which include:
• Assumption of linear elastic behaviour, without considering nonlinearity and plasticity
• Design factors of safety (e.g., for load actions and for material properties)
• Boundary conditions and joint fixities (e.g., support restraints, degree of partial composite action)
• Effects of load distribution
• Contribution of secondary elements (e.g., parapets, asphalt surfacing layers, infill materials)
• Live load model

In order to relax or modify these potentially conservative assumptions in a safe manner and thus facilitate the evaluation of more accurate and reliable assessment ratings, the following are needed: (1) more realistic modelling and analysis methods (e.g., modelling of nonlinearity and plasticity), (2) more accurate and reliable information on model parameters (e.g., material, geometry, boundary conditions, loading), and (3) better knowledge of in-service structural behaviour (e.g., load path, load distribution). In current industry practice, bridge inspection and testing data can be used to facilitate more realistic bridge assessment. However, the adoption of bridge monitoring technologies and the use of bridge monitoring data are still very limited in practice.

1.2 Research context: bridge monitoring and model updating

To improve bridge O&M, new materials, technologies and processes are continually evolving. One technology is structural health monitoring (SHM), which aims to improve asset performance by measuring and learning from in-service structural behaviour. To investigate the manner in which bridge monitoring systems are currently utilised, Webb et al. (2015) conducted a comprehensive literature survey of existing bridge monitoring deployments and developed a classification framework with five categories defining the reasons why a bridge monitoring system is deployed. These are summarised in Table 1.2. Figure 1.1 provides a summary of the bridge SHM installations examined. It can be seen that of the 45 installations examined, only five demonstrated clear benefit to the bridge owners. Realising the practical value of bridge SHM to bridge O&M remains a key challenge to both researchers and practitioners.
Table 1.2 Categories of bridge SHM.

<table>
<thead>
<tr>
<th>Category from Webb et al. (2015)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sensor deployment studies</td>
<td>Demonstrate the ability of new sensor technologies (e.g., accuracy and precision) for measuring certain parameters of interest</td>
</tr>
<tr>
<td>Anomaly detection</td>
<td>Detect any anomalous structural behaviour which deviates from a predetermined performance baseline using a data-driven (i.e., model-free) approach</td>
</tr>
<tr>
<td>Model validation</td>
<td>Validate and update a structural model by relating sensor measurements to model predictions and interpreting any model-data discrepancies</td>
</tr>
<tr>
<td>Threshold check</td>
<td>Check sensor measurements against certain threshold levels set by engineering knowledge, experience or prediction, beyond which certain actions may be taken</td>
</tr>
<tr>
<td>Damage detection</td>
<td>Identify the type, location, extent and rate of any structural damage on a bridge structure</td>
</tr>
</tbody>
</table>

Fig. 1.1 Bridge SHM deployments by category and number of deployments which demonstrated clear benefit to bridge O&M (adapted from Webb et al. (2015)).
In particular, of these five categories, model validation was found to be the most common reason for deploying an SHM system, however, none of these specific installations demonstrated clear benefit to bridge O&M. Model updating is commonly used as part of the model validation process in research to integrate monitoring data and structural modelling to create an accurate “as-is” analysis model. In general, monitoring and modelling represent two sources of information which engineers use to better understand the real performance of bridges. The former aims to capture the in-field structural response, operational loading, environmental conditions and physical properties; while the latter, most commonly based on the finite element method (FEM) or grillage method, aims to simulate the underlying engineering physics such as material behaviour, structural mechanics and soil-structure interaction. How to relate these two sources of information together, i.e., the integration of monitoring and modelling, to reconcile the observed structural behaviour or change of behaviour remains a key challenge for bridge applications. The key difficulties lie in the complexities and uncertainties of bridges in operation (e.g., structural and material imperfections, uncontrolled environmental and operational conditions, uncertain boundary conditions), which make both their monitoring and modelling susceptible to numerous sources of uncertainty.

More recently, there has been a vision of developing bridge digital twins as virtual representations of the physical bridge assets, which have the following key characteristics:

1. They serve as virtual simulation models which can be updated continuously as new data (e.g., monitoring data) becomes available.

2. They are connected to the physical assets to provide real time information (e.g., structural condition) and enable remote management.

3. They can be used to perform “what-if” scenarios for predicting asset performance and facilitating proactive maintenance.

Model updating (i.e., the integration of monitoring and modelling) is an important part of the digital twinning process to create virtual simulation models that closely represent the behaviour and performance of the physical bridge assets.
1.3 Research motivation and question

It was a shock when I began to appreciate that there was an enormous gulf between what goes on in the research laboratory and what goes on in practice, especially in terms of design, use of codes of practice, that sort of thing.

— Malcolm Bolton (2020)

Despite over three decades of research on bridge monitoring and over two decades of research on bridge model updating, there has been little sign of industry uptake by bridge practitioners. Based on the contexts provided in the previous sections (1.1 and 1.2), the general motivation of this work is summarised as follows:

• We need to define and demonstrate the practical value of bridge monitoring more clearly and specifically.

• We need better ways of relating monitoring data to model predictions to produce results that can be interpreted, validated and used by the practicing bridge engineers.

• We need better bridge assessment tools to address the current mismatch between assessment ratings and actual capacities of bridges and thus facilitate more efficient utilisation of existing bridge assets.

The overall aim of this research is to investigate how monitoring data can be used to improve the estimates of structural capacity and structural utilisation of bridges. More specifically, the research question is summarised as follows:

How can monitoring data be utilised to improve bridge assessment by better understanding and quantifying: (1) commonly made assumptions about structural behaviour in bridge modelling and analysis, and (2) the in-service structural utilisation and “margin of capacity” of bridges?

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In addition, this PhD thesis provides a way forward to the following question about “margin of capacity”, which is commonly posed by many bridge owners:

_How much additional live load can be placed on a bridge before violating its safety, serviceability and long-term durability criteria?_

### 1.4 Thesis outline

To address the research motivation and question in section 1.3, this PhD thesis consists of two parts. In the first part (Chapter 3), a series of in-depth industry interviews were conducted with expert professionals in bridge O&M to examine why there had been little adoption of bridge monitoring and model updating in practice, despite decades of research. In the second part (Chapters 4 to 8), based on the findings from the literature review and the industry interviews, a systematic approach to “monitoring-informed bridge assessment” was developed (specifically, using strain monitoring data to enable more realistic bridge modelling and analysis and hence more realistic structural capacity and utilisation assessment). Different aspects of structural behaviour characteristics and in-service structural utilisation were investigated, with a focus on different potentially conservative assumptions commonly made in bridge assessment (as summarised in section 1.1). The developed methods were applied to the Chebsey bridge, an operational prestressed concrete railway bridge in Staffordshire, U.K.

The outline of this PhD thesis is summarised as follows:

- **This chapter** provides the general context of this PhD research and states the research motivation and objective.

- **Chapter 2** reviews previous research on using monitoring data to improve structural modelling of bridges (in particular, structural model updating of bridges). To facilitate the investigation of bridge structural assessment using monitoring data, a review of existing research on how monitoring data can inform bridge assessment is also presented.

- **Chapter 3** presents the findings from the industry interviews on the adoption of monitoring, modelling and model updating in bridge O&M activities, and in particular, the disconnects between research and practice regarding bridge monitoring and model updating.

- **Chapter 4** presents an overview of the monitoring programme for the Chebsey bridge, an operational prestressed concrete railway bridge in Staffordshire, U.K., which was
instrumented with a dense network of fibre optic sensors (FOS) for measuring strain and temperature changes. The monitoring of this bridge serves as the case study for the research investigations on using monitoring data to improve bridge capacity prediction and utilisation assessment (Chapters 5 to 8).

- **Chapters 5 and 6** focus on monitoring and evaluating structural behaviour to enable more realistic structural modelling and analysis and thus more realistic capacity assessment of bridges.

- **Chapter 5** focuses on monitoring and evaluating prestress loss behaviour, which is a critical performance indicator for prestressed concrete bridges. Code predictions (based on Eurocode 2 and AASHTO bridge design specifications) and sensor measurements of prestress losses were compared and interpreted.

- **Chapter 6** focuses on monitoring and evaluating structural behaviour under live load. Specifically, load distribution characteristics, contribution of secondary elements and support boundary conditions were investigated. New or improved methods were developed to evaluate these structural behaviour characteristics using real-time strain monitoring data under operational conditions. A grillage model was created and then updated based on both sensitivity analysis and monitored structural behaviour characteristics.

- **Chapter 7** focuses on monitoring, evaluating and visualising in-service structural utilisation to better understand the “margin of capacity”, which is defined in this study as how much additional live load can be safely placed on a bridge without violating the design performance criteria. Three definitions of structural utilisation and three types of visualisation were proposed for different use cases. In particular, the study has demonstrated how strain-based monitoring can be used to measure certain different load effects, which can then be compared with the corresponding design load effects and thus inform the “margin of capacity”.

- **Chapter 8** focuses on evaluating the uncertainty levels of the output parameters of interest (e.g., structural properties, load effects) in the aforementioned studies on structural behaviour and structural utilisation. Different uncertainties associated with sensor data and data processing were analysed and then propagated to the output parameters of interest. In addition, a sensitivity analysis was performed to identify the sources of data-related uncertainty that are most significant.
Chapter 9 provides the conclusions of this PhD thesis, highlights the primary contributions and discusses avenues of future research.

1.5 Principal publications

1.5.1 Journal papers by the author


1.5.2 Conference papers by the author


Chapter 2

Literature Review

2.1 Introduction

This chapter first reviews existing research on using monitoring data to improve structural modelling of bridges (i.e., model validation and model updating – the third category of bridge SHM from Webb et al. (2015), refer to section 1.2). Specifically, a systematic literature survey on existing field studies of bridge model updating (i.e., the integration of structural monitoring and structural modelling) is presented. The objectives are to provide an overview of the state of research on bridge model updating deployments and techniques and to examine their potential benefits to bridge O&M. Key questions of interest include “What to measure?”, “What model parameter to update and why?” “How to update?” and “What information can be extracted to inform bridge O&M?”.

To facilitate the investigation of using monitoring data to improve bridge assessment (e.g., through more realistic structural modelling and analysis), existing literature on this topic was reviewed. In the bridge SHM research community, compared with damage detection, assessment of structural capacity and “margin of capacity” is a less well-established field and fewer studies have focused directly on using monitoring data to improve bridge assessment.

2.2 Bridge model updating

2.2.1 Overview of the literature survey

The methodology adopted for this literature survey on field studies of bridge model updating was consistent with those of similar literature surveys in built environment research (Li et al., 2018; Vagnoli et al., 2018; Wang and Kim, 2019; Webb et al., 2015). In particular, to systematically search and select the literature for review, a content analysis-based review
method was adopted (Seuring and Gold, 2012). A number of input keywords were identified to define the scope of relevant literature. These keywords were bridge, monitoring, model updating, structural identification and finite element modelling. It was decided to focus only on case studies published as peer-reviewed technical journal articles. The literature search was facilitated through the use of Scopus and Google Scholar.

Two key selection criteria were used:

1. The above-mentioned keywords or their synonyms should be included in the title or abstract. A brief examination of the content was conducted for each paper to assess the level of relevance.

2. To examine the value of bridge model updating to bridge O&M in practice, the data utilised should be field monitoring data from bridges in operation, rather than test data of scaled bridges in the controlled laboratory environment or simulated data.

Both the model updating methodologies and outputs (in particular, information extracted from the updated model) were examined.

A total of 104 journal papers were identified. It should be noted that while these may not provide full coverage of all relevant papers, they provide a good representation of existing research studies in this field. Figure 2.1 shows the number of papers collected by year of publication. It can be seen that as bridge SHM technologies and model updating techniques have developed over the years, more research papers have been published in this field.

Fig. 2.1 Number of reviewed journal papers on bridge model updating studies by year of publication.
Six overarching questions for bridge model updating have been identified. These represent the decisions that need to be made when implementing bridge model updating in practice:

1. How to construct an appropriate model for updating?
2. What model properties should be updated?
3. What monitoring data can be utilised?
4. What model updating techniques can be used?
5. How to verify and validate the updated model?
6. What information can be extracted from the updated model?

The answers to these six questions depend on the exact applications, and therefore there may not be a one-size-fits-all strategy for bridge model updating. Details of the surveyed journal papers based on these six questions are provided in Appendix A. The findings of the literature survey are summarised in the following subsections (2.2.2 to 2.2.7) under these six questions.

### 2.2.2 How to construct an appropriate model for updating?

A bridge design model, such as a finite element (FE) model, is typically created under simplified and idealised conditions (e.g., rigid joints, homogeneous material, perfect alignment). The idealised model may serve as a baseline for engineering design. However, it has been found to be challenging to generate an appropriate structural model for the purposes of performing model updating and supporting bridge O&M. On the one hand, the model needs to be sophisticated enough to describe the structural behaviour or diagnose any structural damage of interest. On the other hand, the model also needs to be sufficiently simple so that the model updating inverse problem is well-posed (i.e., it has one unique solution which is not highly sensitive to data uncertainties). How to develop an appropriate model for updating is a complex decision to make and depends on many factors such as the monitoring data collected and the end applications.

Overall, it has been found that this question is not often explicitly addressed in the surveyed literature. Some early studies on bridge model updating, which were based on measurements of dynamic properties, found that for the updated model parameters to be physically meaningful, the fidelity (e.g., element type, mesh size) of the initial model should be sufficiently high (Brownjohn and Xia, 2000; Xu and Xia, 2012). Different types of models with different model fidelities have been attempted in different research studies, as summarised below:
• **2D versus 3D**
  Most research studies used 3D models. Some used 2D models for identifying and calibrating boundary conditions (e.g., horizontal and rotational support spring stiffness: $K_h$, $K_r$) and updating beam stiffness (e.g., $EI$) under controlled load tests (Bentz and Hoult, 2017; Okasha et al., 2012).

• **linear versus nonlinear**
  Most research studies used linear models. Some used nonlinear analyses (e.g., taking into account nonlinear material properties) in combination with model updating to perform load capacity assessment or structural reliability analysis (Ding et al., 2012; Okasha et al., 2012).

• **multi-scale model (a.k.a. hybrid model)**
  Some research studies used a multi-scale or hybrid model (Zhu et al., 2015). This is a structural model where different model fidelities (e.g., different element types, different mesh sizes) are used for different parts of the structure. It is adopted to balance between model accuracy and computational efficiency. High model fidelity is applied to certain parts of the bridge which have complex structural behaviour or are of particular interest to the bridge engineer.

• **surrogate model**
  Some research studies used a surrogate model such as a response surface model (Xiao et al., 2015). This is a data-driven model to provide a simple relationship (e.g., polynomial) between input model parameters and output objective functions of interest. This is applied to further reduce the computational cost of the model updating process.

More recently, there has been an increasing amount of research on automated selection of model class in model updating (Kontoroupi and Smyth, 2017; Yuen et al., 2019), although based on the surveyed studies, these developed methods have not yet been attempted for bridge applications.

### 2.2.3 What model properties should be updated?

The discrepancy between model predictions and sensor measurements for a bridge may be the result of a combination of different factors and sources of uncertainty. These have been discussed and summarised in a number of papers (Goulet et al., 2010; Mottershead et al., 2011; Simoen et al., 2015). Table 2.1 provides a summary of these uncertainties. In general, these uncertainties may also be categorised as *epistemic uncertainties* or *aleatory uncertainties*. The former are due to lack of knowledge (e.g., insufficient information on
model parameters), while the latter are due to inherent randomness of the physical system or the sensor system (e.g., variability in structural and material properties, environmental effects, measurement noise).

Table 2.1 A summary of key uncertainties in structural analysis model and structural monitoring data.

<table>
<thead>
<tr>
<th>Model uncertainties</th>
<th>Data uncertainties</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model parameter</td>
<td>Model structure</td>
</tr>
<tr>
<td>Material properties</td>
<td>Modelling assumptions and simplifications</td>
</tr>
<tr>
<td>Geometric properties</td>
<td>Discretisation (e.g., for FE models) and approximations (e.g., for surrogate models)</td>
</tr>
<tr>
<td>Boundary and continuity conditions</td>
<td></td>
</tr>
<tr>
<td>Operational and environmental loading</td>
<td></td>
</tr>
</tbody>
</table>

Based on the surveyed literature, it is common practice to minimise model structure uncertainties first in order to prepare the initial model for bridge model updating. Specifically, this includes selecting the appropriate model type or model class (e.g., which structural components or details to be included, boundary conditions, element type, mesh size). Data uncertainties also need to be addressed before model updating (e.g., data cleansing, data synchronisation, data detrending). Currently, these are achieved primarily by manual examination of the design and modelling assumptions, initial data inspection and interpretation as well as engineering judgement (Bentz and Hoult, 2017; Goulet et al., 2010).

Existing model updating techniques mainly deal with uncertain model parameters. In terms of selecting which parameter(s) to update, a large number of papers adopted the general principle given in Brownjohn et al. (2001), which stated that the selected parameter(s) should satisfy two conditions:
2.2 Bridge model updating

1. The value of the model parameter must be uncertain.

2. Changes of the monitored output response should be sufficiently sensitive to changes of the model parameter.

In many cases, a parametric study (i.e., sensitivity analysis) was performed to assist in the selection of updating parameters. These parameters may then be treated as the unknown parameters in the model updating problem.

2.2.4 What monitoring data can be utilised?

“What should be measured and why?” is a fundamental question raised by many bridge practitioners for bridge SHM. The answer to this question depends on what the bridge manager wants to know and how the SHM data would be interpreted to extract useful information once it is collected.

Overall, based on the surveyed bridge model updating studies, there are two types of monitored structural responses or structural properties which are most commonly used for bridge model updating. One is to use identified modal properties (e.g., modal frequency, mode shape) from the dynamic response, typically obtained using accelerometer data, under ambient or forced vibration tests (Brownjohn and Xia, 2000; Xu and Xia, 2012). In this case, for ambient vibration tests, real-time operational data of ambient responses can be used to avoid the need to disrupt traffic. However, as modal properties usually represent the global condition of a structure, they have generally been found to be relatively insensitive to local change in stiffness due to localised structural change or structural damage (Xu and Xia, 2012).

The other is to use strain or displacement data under controlled load tests where the loading can be measured with relatively high accuracy (Bentz and Hoult, 2017; Okasha et al., 2012). However, controlled load tests would either require bridge closure and thus cause traffic disruption or need to be conducted prior to bridge opening. In addition, a few studies used geometry-based model updating for masonry arch bridges. This approach uses measurement of geometry (e.g., laser scanning for arch geometry) to infer and evaluate permanent deformation, thereby informing the underlying deformation mechanism and detecting any structural damage (Conde et al., 2018).

There are two additional challenges when interpreting bridge monitoring data: (i) data quality, and in particular, whether the sensor data is sufficiently sensitive to detect any structural change or structural damage of interest; and (ii) it may be difficult to distinguish between the effects due to changes of environmental or operational conditions and the effects
due to physical changes of the bridge structure (Farrar and Worden, 2012; Ni et al., 2007; Vagnoli et al., 2018).

Figure 2.2 shows the number of surveyed papers based on the monitoring data utilised. It can be seen that the majority of the existing research is based on modal properties under vibration tests, although recently there have been more studies using changes of strain or displacement under load tests.

![Figure 2.2 Number of reviewed papers per year of publication based on the type of monitoring data utilised.](image)

2.2.5 What model updating techniques can be used?

In SHM research, structural model updating is essentially an inverse problem which updates the model parameters and sometimes other modelling assumptions by matching model predictions with sensor measurements. Based on the surveyed literature, there are four main categories of model updating technique: (i) manual tuning, (ii) residual minimisation, (iii) Bayesian model updating, and (iv) error-domain model falsification. A brief description is provided for each technique as follows:

- **Manual tuning**
  This type of approach involves selecting and updating model parameters and/or modelling assumptions by manually examining the discrepancies between model predictions and sensor measurements. Specifically, reasons behind the discrepancies are first proposed based on engineering knowledge, judgement and/or experience. These are
then verified by manual tuning of the model. Iterative trial-and-error processes are often involved to refine the model until there is a reasonable agreement between model predictions and sensor measurements based on engineering judgement. Other sources of information may also be used, such as material testing and visual examination (e.g., examine support conditions to infer boundary conditions, examine crack patterns to infer structural behaviour mechanism). Example applications include Bentz and Hoult (2017) and Daniell and Macdonald (2007).

**Residual minimisation**

This type of approach involves framing the model updating problem as a multi-variate deterministic optimisation problem to optimise the model parameters. Constrained optimisation (e.g., setting constraints on model parameter values) is often used to ensure that the updated model does not lose physical meaning. The objective function is some measure of discrepancy between model predictions and sensor measurements of structural response. In the case where more than one type of structural response data are used, a weighted sum of the discrepancies for these structural responses is commonly used. How to set the weighting factors for different objectives (e.g., different types of structural response) has not been well articulated in the surveyed literature and thus remains an area of future research. Currently, these weighting factors are often set based on engineering judgement and/or testing of different combinations of weighting factors to compare “goodness of fit” (Xiao et al., 2015). Detailed workflow and example applications can be found in Brownjohn et al. (2001) and Živanović et al. (2007).

**Bayesian model updating**

The Bayesian model updating approach is developed based on Bayes’ theorem: posterior probability density function (pdf) = prior pdf × likelihood function / (integral of prior pdf × likelihood function over the entire parameter space). In the context of model updating: \( p(\text{model parameter} \mid \text{data}) = p(\text{model parameter}) \times p(\text{data} \mid \text{model parameter}) / p(\text{data}) \), where \( p(A) \) is the probability of \( A \) and \( p(A|B) \) is the probability of \( A \) occurring given that \( B \) is true. The prior probability density function shows the prior information of the uncertain model parameters without the SHM data, and the likelihood function reflects the information extracted from the SHM data. The Bayesian approach provides not only the optimal estimates but also the quantification of estimation uncertainty in the form of a probability distribution. The theoretical framework and an example application can be found in Beck and Katafygiotis (1998) and Jang and Smyth (2017), respectively.
Error-domain model falsification

This type of approach involves first generating a pool of candidate models with all possible combinations of model parameter values and then falsifying the models from this pool by performing a threshold check on the discrepancy between model predictions and sensor measurements. The threshold value is set based on the sum of the effects from multiple sources of modelling errors and data errors. The objective is to narrow down the number of candidate models as new monitoring data becomes available. Detailed workflow and example applications can be found in Goulet et al. (2010) and Goulet and Smith (2013).

More recently, there has also been research involving the use of machine learning (ML) based techniques (e.g., Gaussian processes, neural networks) in bridge model updating to identify model parameter values by incorporating a data-driven approach (Gokce et al., 2013; Hasançebi and Dumlupınar, 2013; Soyoz and Feng, 2009; Yin and Zhu, 2020). The data-driven approach is used to characterise the relationship between the output model responses or model properties of interest and the relevant input model parameters.

Figure 2.3 shows the number of surveyed papers based on the main model updating technique used. Specifically, Figure 2.3a shows manual technique versus automated techniques and Figure 2.3b shows different types of automated techniques. Some research papers used a combination of more than one technique, in which case the main technique used is chosen for categorisation purposes. Manual tuning is sometimes applied as a prior step to automated model updating in order to generate an appropriate initial model for further updating. A typical example is the identification of appropriate boundary fixities or degrees of freedom (Bentz and Hoult, 2017; Okasha et al., 2012; Robert-Nicoud et al., 2005). Overall, it can be seen that the majority of existing research is on automated model updating techniques. Of the three automated techniques, residual minimisation is most commonly adopted, and recently there have been more applications of other automated techniques for bridge model updating.

2.2.6 How to verify and validate the updated model?

Based on the surveyed literature, there are two main methods for verifying and validating the updated bridge model. The first method is to use other measurement data (e.g., structural response at different locations on the same bridge, other types of structural response, material properties from material testing) to test whether there is a close match between these other measurements and the corresponding predictions of the updated model. The second method is mainly based on engineering interpretation and judgement (i.e., physical explanation) to
Fig. 2.3 Number of reviewed papers per year of publication based on the model updating technique: (a) manual technique versus automated techniques; and (b) automated model updating techniques (based on the main technique used).
assess whether the associated structural changes based on the bridge model updating make engineering sense. Other methods mentioned in the surveyed literature include checking convergence by updating perturbed models (i.e., models with parameters perturbed about the values of the updated parameters) (Brownjohn et al., 2001) and comparing and checking the consistency of results from different model updating techniques (Weng et al., 2011).

One question which has not been explicitly addressed yet is whether the updated model, if it were to be used for making predictions, is valid for other loading scenarios or ambient conditions of interest. For example, a model updated using monitoring data under small load cases (e.g., normal traffic loading, normal weather conditions) may not necessarily be valid for extreme load cases (e.g., abnormal vehicle loading, severe wind loading, earthquake loading).

Figure 2.4 shows the percentage of each model verification and validation method adopted in the surveyed literature. It can be seen that the majority of these studies have not specifically mentioned model verification and validation. Around a third of the papers used other measurement data, and engineering interpretation and judgement is not very often used.

Fig. 2.4 Percentage of each model verification and validation method adopted in the reviewed papers.

### 2.2.7 What information can be extracted from the updated model?

Based on the literature survey presented in this chapter and the industry interviews presented in the next chapter (Chapter 3), five potential capabilities have been identified, which
are related to bridge monitoring and model updating, and can be useful to bridge O&M (particularly bridge condition appraisal). These are:

1. Damage detection
2. Damage criticality evaluation
3. Reserve load capacity assessment
4. Remaining service life prediction
5. “What-if” scenarios simulation

The surveyed papers were examined based on these five categories to identify the information extracted from bridge model updating. Figure 2.5 shows the percentage of each category of information extracted from the surveyed bridge model updating studies. Some studies had more than one type of output information, in which case the main type is chosen for categorisation purposes.

![Fig. 2.5 Percentage of each category of bridge O&M related output information (actual or intended) from bridge model updating studies in the reviewed papers.](image)

It can be seen from Figure 2.5 that of the papers which specified the outputs of model updating for bridge O&M, the two most common ones are damage detection and load capacity assessment. Some research coupled model updating with other analyses such as structural reliability analysis (Gokce et al., 2013; Okasha et al., 2012) and fragility functions (Li et al., 2013). The surveyed papers on remaining service life prediction and damage criticality
evaluation are mostly based on fatigue analysis of critical bridge elements (Lee and Cho, 2016; Pasquier et al., 2014).

As for damage detection, the surveyed bridge model updating studies on this topic often rely on the assumption that localised damage results in local reduction in stiffness, which can then be detected from sufficient change of structural behaviour (most commonly using changes of modal properties). However, most of the research did not specify the exact types of damage that can be detected. A few investigated specific types of structural damage, which include:

- **Boundary conditions**
  e.g., pier settlement (Teughels and De Roeck, 2004), support stiffness reduction due to scour (Garcia-Palencia et al., 2015), horizontal stiffness of expansion joints (Xia et al., 2020)

- **Significant section loss**
  e.g., artificially introduced torch cuts to girders (Perera and Ruiz, 2008), steel corrosion of steel truss bridges (Jang et al., 2013)

- **Cable damage**
  e.g., cable slack of cable-stayed bridges (Degrauwe et al., 2009)

- **Cracking**
  e.g., crack pattern of masonry arch bridges (Conde et al., 2018)

Most of the surveyed studies, especially early ones, used modal frequencies and mode shapes to perform model updating and damage detection, which were generally not sensitive to local damage. More recently, there have been attempts at using potentially more damage-sensitive measurements or features: e.g., mid-span displacement and strain (Jesus et al., 2019), damping (Mustafa et al., 2018).

As for structural assessment, particularly load capacity assessment, some surveyed studies investigated a number of assumptions in structural modelling and analysis of bridges, including:

- **Boundary conditions**
  Bentz and Hoult (2017); Brownjohn et al. (2003); Gokce et al. (2013); Goulet et al. (2010)

- **Material stiffness**
  Bentz and Hoult (2017); Conde et al. (2017); Goulet et al. (2010)
2.3 Using monitoring data to inform bridge assessment

- **Load distribution**
  e.g., lateral live load distribution factor (Dong et al., 2020)

- **Geometry**
  predominantly for masonry arch bridges (Conde et al., 2017)

- **Stiffness contribution of secondary elements**
  e.g., guardrails and safety curbs (Brownjohn et al., 2003; Goulet et al., 2010; Sanayei et al., 2012), fill materials of masonry arch bridges (Conde et al., 2017)

In addition, load testing has sometimes been used to facilitate bridge assessment (Bentz and Hoult, 2017; Sanayei et al., 2012; Zhou et al., 2012).

2.3 Using monitoring data to inform bridge assessment

The most commonly cited area of focus for studies in bridge SHM research has been on damage detection. Fewer studies have been focused directly on using monitoring data to improve bridge capacity assessment, thereby addressing the current mismatch between load rating and actual capacity of bridges (as mentioned in chapter 1). To achieve this, it is important to have a better understanding of in-service structural behaviour and structural utilisation in order to enable more realistic structural modelling and analysis for assessment purposes. Key areas of interest in structural behaviour include load distribution, boundary and continuity conditions, contribution of different structural components, etc.

With regards to load distribution (e.g., transverse load distribution across different girders), the design values of load distribution factors (DF) in codes and standards (American Association of State Highway and Transportation Officials [AASHTO], 2017; Highways England, 2019) are often based on experimental studies of small-scale model bridge systems (e.g., grillage beam systems), such as those conducted in Morice and Little (1955). To investigate the live load distribution of bridges in service, a number of research studies used strain measurements, typically from strain gauges, under load testing to evaluate DFs and then compared measured DF values with design DF values (Algohi et al., 2019; Barr et al., 2001; Huang et al., 2004; Huseynov et al., 2017). It was found that the degree of conservativeness of the design DF values varied depending on the bridge configurations. More recently, with the development of more advanced sensing technologies, Van Der Kooi et al. (2018) and Lin et al. (2019) demonstrated the strain sensing capabilities of distributed and discrete fibre optic sensor (FOS) systems, respectively. Each of these two studies instrumented an operational railway bridge and characterised its strain distribution, member behaviour and connection behaviour under passing train loads.
As for monitoring boundary and continuity conditions, Bakht and Jaeger (1992) performed an ultimate load test on a short-span bridge with steel girders and concrete deck slab. They found that the actual bending moments observed in the deck were reduced by the bearing restraint forces at the supports. They also found that any composite action that existed between slab and girder under service load, which was inferred from the measured neutral axis position, started to break down as the load approached the failure load. Webb et al. (2014) investigated the fixity of roller bearings of a deteriorated prestressed concrete road bridge by examining whether the bearings underwent the expected longitudinal movements with temperature variations. Hester et al. (2019) demonstrated how bearing movement monitoring data under ambient vibration tests and static load tests can be used to calibrate support boundary conditions in the structural model. Huseynov et al. (2017) used strain data under load testing to investigate the influence of support boundary conditions on transverse load distribution across different girders of an aging road bridge. Hou et al. (2020) used long term strain monitoring data on a road bridge to investigate the assumption of the degree of partial composite action (DCA) between slab and girder in the bridge assessment. Strain measurements across the deck section were used to estimate the neutral axis position and infer the DCA. This in turn was used to evaluate the real strain profiles across the deck and thus enable more accurate assessment of long-term deck durability.

For prestressed concrete bridges, a critical indicator of their structural performance is the remaining level of prestress in the prestressed concrete beams. Several studies have been conducted for interpreting the measured prestress losses of prestressed concrete girders by relating sensor measurements to physics-based models. Webb et al. (2017) performed a direct comparison between the strain measurements from fibre optic sensors along the prestressing strands and the predictions from two different empirical models of prestress loss considering creep and shrinkage (based on Eurocode 2 and research from Collins and Mitchell (1997)). They found a good agreement between the predictions and the measurements; and the discrepancies were smaller in magnitude than the uncertainty level of sensor measurements. Abdel-Jaber and Glisic (2019) presented a systematic approach for uncertainty analysis of FOS measured and code predicted prestress losses to enable more rigorous comparison between the two. They found that although code predictions and sensor measurements of prestress loss were of comparable magnitude in their case study, code predictions were not necessarily overly conservative as often perceived. Huang et al. (2018) presented a finite element model updating approach, informed by parametric studies and engineering judgement, for relating model predictions to sensor measurements of vertical deflections and observed crack patterns of a prestressed concrete bridge. Using the updated FE model predictions, they were able to estimate the realistic creep and shrinkage levels and prestress
losses. Sousa et al. (2014) performed parameter identification by fitting EC2 models of prestress losses to real strain measurements from a prestressed concrete bridge. They also performed load tests to validate and update the FE model of the bridge.

As for case studies of using monitoring data to inform structural modelling of bridges for assessment purposes, Vernay et al. (2018) used an error-domain model falsification approach to calibrate model parameter values and enable worst-case load capacity assessment. The model falsification approach was previously described in section 2.2.5. Lin et al. (2019) compared dynamic FOS measurements of a railway bridge under passing train loads with predictions from a 3D FE model and then investigated the effects of various modelling assumptions (e.g., load distribution, connection fixity and secondary element contribution). Cocking et al. (2020) investigated an existing deteriorated masonry arch bridge and evaluated the outputs from a series of linear-elastic FE models (with different modelling simplifications) against strain monitoring data under train loads. The study quantified the relative contributions of different structural components to the global structural behaviour, which can be used to inform the structural modelling practice for ultimate limit state (ULS) and serviceability limit state (SLS) assessment.

As for structural utilisation, there are some previous research studies on structural utilisation of buildings (Moynihan and Allwood, 2014; Orr et al., 2019). These studies mainly investigated the issue of overdesign, with a focus on design efficiency and design practice. They found that $E_d/R_d$ (where $E_d$ is the design load effect and $R_d$ is the design resistance) in the current design practice for buildings was in general around 50%. Lin et al. (2019) and Davila Delgado et al. (2018) evaluated and visualised the “live load utilisation” (defined in the papers as $\frac{\text{measured live load effect}}{\text{design live load effect}}$) of two instrumented girders of a railway bridge under train-passage events, respectively. To the best of the author’s knowledge, no systematic study on in-service structural utilisation of bridges has been conducted before.

Overall, unlike damage detection, systematic approaches to monitoring informed bridge assessment are less developed in bridge SHM research. There is still a research challenge on how to use structural monitoring data to enable more accurate and reliable structural modelling and analysis in order to enable more realistic assessment of structural capacity and “margin of capacity” (how much additional live load can be safely placed) for bridges. In particular, there is a gap in knowledge on how to use monitoring data, in a safe and systematic way, to modify potentially conservative assumptions in bridge assessment through a better understanding of in-service structural behaviour and structural utilisation.
2.4 Summary

This chapter has first provided a comprehensive literature survey of bridge model updating field studies (i.e., integration of structural monitoring and modelling for bridges). The goal was to examine existing research capabilities and gaps against six overarching questions for bridge model updating, which correspond to six decisions that need to be made when implementing bridge model updating:

1. How to construct an appropriate model for updating?
2. What model properties should be updated?
3. What monitoring data can be utilised?
4. What model updating techniques can be used?
5. How to verify and validate the updated model?
6. What information can be extracted from the updated model?

The chapter then reviewed existing bridge SHM literature on the use of monitoring data to inform structural assessment of bridges, with a focus on (i) typical structural behaviour assumptions in structural modelling and analysis such as load distribution, boundary and continuity conditions and contribution of secondary elements, and (ii) structural utilisation.

A summary on the key capabilities and limitations of existing research is provided as follows:

- **On bridge model updating research methodologies**
  The general research approach of structural model updating is essentially an inverse problem, which updates the model parameters (particularly stiffness parameters) and sometimes other modelling assumptions by matching model predictions with sensor measurements. Based on the literature survey, there are four common updating techniques: manual tuning (e.g., Bentz and Hoult (2017), Daniell and Macdonald (2007)), residual minimisation (e.g., Brownjohn et al. (2001), Živanović et al. (2007)), Bayesian model updating (e.g., Beck and Katafygiotis (1998), Jang and Smyth (2017)) and error-domain model falsification (e.g., Goulet et al. (2010), Goulet and Smith (2013)).

  Regarding the monitoring data utilised, most studies used measured dynamic properties (e.g., modal frequency, mode shape) to perform model updating (e.g., Brownjohn and Xia (2000), Xu and Xia (2012)), although they have generally been found to be insensitive to local stiffness reduction due to localised damage (Xu and Xia, 2012).
Some studies used strain or displacement measurements under controlled load tests (e.g., Bentz and Hoult (2017), Okasha et al. (2012)), however, this would require bridge closure in practice.

- **On bridge model updating research outputs**
  A majority of the surveyed studies did not specify what useful information can be extracted from the updated model in order to improve bridge O&M. Of the studies which specified the bridge O&M related objectives of the model updating exercise, the most commonly cited area of focus has been on damage detection, using the assumption that a localised structural damage results in a local reduction in structural stiffness. However, most of these damage detection studies did not specify the exact types of damage that can be detected. Compared to damage detection, fewer studies have been focused directly on using monitoring data to improve structural capacity assessment, and to the best of the author’s knowledge, no systematic approach for this has been developed.

- **On monitoring structural behaviour to inform bridge assessment**
  1. **On load distribution:** Existing research on measuring load distribution mostly used strain data under controlled load tests or monitored maximum strain response under live load (Algohi et al., 2019; Barr et al., 2001; Huang et al., 2004; Huseynov et al., 2017). The former is predominantly a one-off exercise which requires bridge closure, while the latter is particularly susceptible to strain measurement noise or anomalies. Overall, there is still a research gap on how to use real-time strain monitoring data to evaluate load distribution continuously and automatically, with high accuracy and reliability (i.e., low uncertainty level), and under live load response (i.e., normal operational conditions with no bridge closure or special access requirements).

  2. **On boundary conditions:** Existing research on monitoring support boundary conditions requires either direct measurement of support movement (e.g., bearing movement) or strain measurements of the bridge deck under controlled load tests (Bakht and Jaeger, 1992; Hester et al., 2019; Huseynov et al., 2017; Webb et al., 2014). The former requires special access to the bridge supports (e.g., bearings), and the installation of the sensors required may be difficult. The latter requires bridge closure, and it is predominantly a one-off exercise. Sometimes both techniques have been used to improve the level of confidence in the results. Overall, there is still a research gap on how to use real-time strain monitoring data to evaluate support boundary conditions continuously and automatically,
with high accuracy and reliability (i.e., low uncertainty level), and under live load response (i.e., normal operational conditions with no bridge closure or special access requirements).

3. **On contribution of secondary elements to structural stiffness**: Existing research used comparisons between sensor measurements and model predictions of strain response to infer the stiffness contribution of secondary elements (e.g., parapets, fill materials), which is often due to the partial composite action (i.e., the degree of fixity) between the primary structural elements (e.g., main girders, masonry arches) and these secondary elements (e.g., Bakht and Jaeger (1992), Hou et al. (2020)). This contribution may be used for SLS assessment but may not necessarily be valid for ULS assessment as some research has found that the degree of partial composite action may start to break down as the loading approaches ULS.

4. **On prestress loss (for prestressed concrete bridges)**: There have been very few field studies which have measured prestress loss evolution in prestressed concrete bridges (e.g., Webb et al. (2017), Abdel-Jaber and Glisic (2019)). Consequently, the nature of the progression of prestress losses in real world bridge applications is largely unknown and any resulting data comparing actual losses versus code predictions has rarely been available.

- **On monitoring structural utilisation to inform bridge assessment**
  Few research studies in bridge SHM have been focused on using monitoring data to evaluate in-service structural utilisation and thus inform the “margin of capacity” (how much additional live load can be safely placed on a bridge) (Davila Delgado et al., 2018; Lin et al., 2019). Unlike damage detection, this is less developed as a research field in bridge SHM. In particular, there is no systematic approach to or workflow on how to use monitoring data to safely modify potentially conservative assumptions in bridge assessment in order to address the mismatch between load rating and actual capacity, thereby unlocking additional capacities for bridge assets.
Chapter 3

Industry Interviews

3.1 Introduction

This chapter investigates the reasons behind the limited industry uptake of bridge monitoring, modelling and model updating (i.e., the integration of monitoring and modelling) in bridge operation and maintenance (O&M) activities, and in particular, the challenges of implementing these technologies and techniques in practice. A series of industry interviews with 19 expert bridge professionals were undertaken for the investigation. This was important for identifying the disconnects between research and practice in these areas and the additional research needed to drive industry adoption of bridge monitoring and model updating in order to realise their practical value to bridge O&M.

3.2 Methodology

Seventeen face-to-face semi-structured interviews were conducted with nineteen expert bridge professionals in the U.K. (10 bridge owners/operators and 9 bridge consultants; two of the interviews had 2 interviewees). The interviewees were carefully selected to be representative of those involved in bridge O&M activities in the U.K. The interviewed group represented a broad range of bridge O&M activities, including owners, operators and consultants, for both highway and rail operations and covering all levels of operation scope: e.g., national, regional/county, local authority. All interviewees have a technical background in civil and structural engineering and at least ten years’ experience in bridge O&M activities. This was to ensure that the interviewees had sufficient expertise to provide insightful answers to the questions. Details of the interviewees are presented in Table 3.1.
### Table 3.1 Details of the interviewees.

<table>
<thead>
<tr>
<th>Interview</th>
<th>Role</th>
<th>Sector</th>
<th>Scope</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>Highway sector lead</td>
<td>Highways</td>
<td>National</td>
</tr>
<tr>
<td>C2</td>
<td>Senior bridge engineer</td>
<td>Highways &amp; Rail</td>
<td>Regional</td>
</tr>
<tr>
<td>C3</td>
<td>Principal structures advisor</td>
<td>Highways</td>
<td>National</td>
</tr>
<tr>
<td>C4</td>
<td>Principal engineer</td>
<td>Rail</td>
<td>National</td>
</tr>
<tr>
<td>C5</td>
<td>Head of bridge engineering</td>
<td>Highways &amp; Rail</td>
<td>National</td>
</tr>
<tr>
<td>C6</td>
<td>Head of bridges &amp; structures</td>
<td>Highways</td>
<td>Local authority</td>
</tr>
<tr>
<td>C7</td>
<td>Project manager</td>
<td>Highways</td>
<td>Local authority</td>
</tr>
<tr>
<td>C8</td>
<td>Major bridge manager</td>
<td>Highways</td>
<td>Regional</td>
</tr>
<tr>
<td>C9</td>
<td>Independent consultant</td>
<td>Highways &amp; Rail</td>
<td>Regional</td>
</tr>
<tr>
<td>C10</td>
<td>Independent consultant</td>
<td>Highways &amp; Rail</td>
<td>Regional</td>
</tr>
<tr>
<td>C11</td>
<td>Professor (consultant)</td>
<td>Highways &amp; Rail</td>
<td>National</td>
</tr>
<tr>
<td>C12</td>
<td>Professor (consultant)</td>
<td>Highways &amp; Rail</td>
<td>National</td>
</tr>
<tr>
<td>C13</td>
<td>Head of profession</td>
<td>Highways &amp; Rail</td>
<td>National</td>
</tr>
<tr>
<td>C14</td>
<td>Head of structures policy</td>
<td>Highways</td>
<td>Regional</td>
</tr>
<tr>
<td>C15</td>
<td>Instrumentation &amp; monitoring lead</td>
<td>Highways</td>
<td>National</td>
</tr>
<tr>
<td>C16</td>
<td>Head of structures</td>
<td>Highways</td>
<td>Regional</td>
</tr>
<tr>
<td>C17</td>
<td>Bridge master</td>
<td>Highways</td>
<td>Local authority</td>
</tr>
<tr>
<td>C18</td>
<td>Major bridges manager</td>
<td>Highways</td>
<td>Regional</td>
</tr>
<tr>
<td>C19</td>
<td>Technical director</td>
<td>Highways</td>
<td>Regional</td>
</tr>
</tbody>
</table>

The methodology adopted was consistent with those of similar interview studies in built environment research (Baker et al., 2017; Bennetts et al., 2019; Dadzie et al., 2018; Gardner et al., 2018). In this study, the interviews were chosen to be semi-structured to allow for targeted and in-depth analysis of how bridge monitoring, modelling and model updating could be implemented in practice to improve the maintenance of bridges, under the broad context of the day-to-day practice and decision making in bridge O&M. Specifically, the interviews examined the following six themes:

1. Current practice for bridge damage detection and structural assessment
2. Current practice for bridge monitoring and modelling
3. Key structural components and issues that keep bridge practitioners awake at night
4. Key capability gaps in bridge condition appraisal
5. Barriers and incentives to using bridge monitoring and modelling in practice
6. Industry perspectives on bridge model updating
The main questions used in the interviews are presented in Appendix B. The interviews were recorded, transcribed and then analysed by “coding” against these six themes, which consisted of highlighting snippets of each interview that are related to each theme (Saunders et al., 2009).

The validation of the interviews followed the principles and methods for qualitative data analysis given in Brinkmann and Kvale (2014). Firstly, the representativeness of the interviewees was checked, as described previously and shown in Table 3.1. Secondly, for five of the six themes (1 to 5) examined, consensus or majority views among the interviewees were distilled to ensure a sufficient degree of reliability for the interview findings (as reported in sections 3.3 to 3.5). Where there was a major difference in opinion on an important issue (which mainly applies to damage detection under theme 4 (“Key gaps in capability in bridge condition appraisal”), as reported in section 3.4.2), this difference was highlighted and all opinions were included. The objective of theme 6 (“Industry perspectives on bridge model updating”) was to gather valid comments and questions raised by the expert bridge professionals (as reported in section 3.5.2). Thirdly, the distilled interview findings were sent back to a subset of interviewees for checking and feedback.

Finally, it should be noted that while this qualitative interview research was set in the U.K. context of bridge O&M practice, the findings may be transferred to similar bridge O&M situations around the world. Specifically, the study provides insights into the types of questions and issues that can be raised by bridge practitioners worldwide as well as their perspectives on bridge model updating. Specific descriptions of the context of this interview research (themes 1 and 2) are provided section 3.3. This is to enable the reader to judge to what extent the findings may be generalised in a new situation.

3.3 Industry practice

3.3.1 Current practice for bridge damage detection and structural assessment

Based on the interviews, there are currently two major types of activities identified for bridge condition appraisal in bridge O&M:
1. **Damage detection**: detection and evaluation of bridge damage and deterioration by means of inspection, testing or monitoring.

2. **Structural assessment**: evaluation of reserve load capacity by means of structural assessment of bridges, which typically involves some type of structural analysis and modelling.

According to the majority of interviewees, the decision of whether or not to close (or partially close) a bridge, which is a main concern for bridge owners, is governed by concern for the safety of people within the vicinity of the bridge. This is mainly determined by whether the bridge has sufficient reserve load capacity.

### Damage detection

According to all the interviewees, there are currently two main ways in which damage and deterioration of a bridge can be detected in practice. These are described in Table 3.2. The use of testing and monitoring in practice is mostly reactive rather than proactive. They are undertaken in a targeted manner to investigate and examine a known issue identified by inspections rather than to detect new damage.

### Structural assessment

Compared with repair work and visual inspection, structural assessment is currently not a high priority for many bridge owners and operators in the U.K. According to the majority of interviewees, it is conducted only when required (e.g., driven by immediate and targeted concerns). In the U.K., it is typically conducted once every 18 years and is mainly for the purpose of load capacity assessment (Griffin and Patro, 2018; Highways England, 2019). An extensive program of bridge assessment was carried out in the 1990s when the legal load limit on lorries permitted on U.K. roads was increased from 38 tonnes to 40 tonnes.

Overall, there are three levels of assessment for both highway and railway bridges in the U.K. (Highways England, 2019; Network Rail, 2018). These are summarised in Table 3.3. Most bridge assessment follows a similar procedure, which starts from Level I assessment and then proceeds to higher levels of assessment (e.g., line beam method to grillage method to finite element method) until the evaluated bridge capacity is satisfactory or else actions are deemed necessary to ensure structural safety of the bridge. As for engineering assumptions made in a bridge assessment, three main sources of information are used: codes and standards, inspection, and testing. Other factors to consider in bridge assessment include the age of the
### Table 3.2 Two main ways for detecting bridge damage in U.K. practice.

<table>
<thead>
<tr>
<th>Method</th>
<th>Description and comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge inspection</td>
<td>Ongoing damage and deterioration of bridges are predominantly detected through standard visual inspection regimes and very rarely from monitoring systems. Routine visual inspection records should observe bridge defects, typically in terms of type, location, extent, severity, and possibly cause (based on engineering judgement and investigation) (<a href="#">Highways England, 2021</a>). These current practices are not automated and do not provide detection of damage in real time. In addition, the effectiveness of these inspections relies on bridge inspectors being competent and consistent in carrying out their inspections. Visual inspections have been found to be subjective and inconsistent (<a href="#">Lea and Middleton, 2002; Moore et al., 2001</a>). In particular, hidden defects are very difficult to inspect and detect.</td>
</tr>
<tr>
<td>Reporting by the public and other reporting</td>
<td>Another main source of bridge condition information comes from the general public, police or managing agents. For example, every single Network Rail bridge in the U.K. has a telephone number displayed on site which the public can call to report any observed damage or incidents (e.g., pieces of loose concrete, concrete falling off from the bridge, bridge strike).</td>
</tr>
</tbody>
</table>
bridge structure, original design loading, actual structural behaviour and potential failure modes.

Table 3.3 Three levels of bridge assessment used in U.K. practice.

<table>
<thead>
<tr>
<th>Level</th>
<th>Description and comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Simple structural analysis methods, with conservative assumptions for material properties (i.e., using code values)</td>
</tr>
<tr>
<td>II</td>
<td>Refined structural analysis methods, such as non-linear or plastic analysis methods</td>
</tr>
<tr>
<td>III</td>
<td>Less conservative assumptions for material properties and bridge loading are used, based on measurements (e.g., using worst credible strength (WCS) values based on measured material properties from testing samples), live traffic loading data – bridge specific assessment live load models (BSALL)</td>
</tr>
</tbody>
</table>

Less conservative values for certain parameters may be used for Level III assessment based on measurements and condition survey. Based on the interviews, frequently mentioned examples of these less conservative assumptions are summarised in Table 3.4.

3.3.2 Current practice for bridge monitoring and modelling

Bridge monitoring

All interviewees agreed that overall, very few bridges have real time SHM systems in place in current practice of the U.K. Of the limited number of installations, the majority were on existing bridges as a tool for further investigation and examination of a known defect or issue. Before each bridge SHM system is installed in practice, a value case needs to be made to justify the associated cost and effort. Examples of the most common and useful type of bridge monitoring installation in U.K. practice have been identified based on interviewees’ responses. These are summarised in Table 3.5. One key issue for SHM of existing bridges, as raised by the majority of interviewees, is the understanding of the pre-existing conditions when the monitoring is first deployed (e.g., existing stress, existing number of wire breaks, cumulative displacement of bearings).

Bridge modelling

According to the interviewees, structural modelling, particularly FE modelling, is rarely used in bridge O&M activities in the U.K. It is predominantly undertaken as a one-off exercise after
Table 3.4 Examples of less conservative assumption used for Level III bridge assessment in the U.K.

<table>
<thead>
<tr>
<th>Assumption</th>
<th>Examples and comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material properties</td>
<td>• These are obtained from material testing: e.g., compressive testing of sample concrete cores. The testing results are typically used in a prescribed way using the worst credible strength approach.</td>
</tr>
</tbody>
</table>
| Section geometry         | • This is obtained from measurement of dimensions: e.g., web thickness by electronic thickness gauge, arch barrel thickness by drilling cores and taking measurements, concrete cover by covermeters.  
  • One key issue raised by many interviewees is the quantification of concrete corrosion, in particular, corrosion of reinforcement bars or prestressing tendons inside concrete. This is an important piece of information in bridge assessment and is difficult to obtain. The current practice for determining the remaining amount of reinforcement bars or prestressing tendons is by exposing them, performing visual inspection and where possible, measuring loss of section on specific sample areas. |
| Boundary conditions      | • This is determined qualitatively by visual examination of bearing and joint conditions. However, it is very difficult to quantify the stiffness and restraining effect of these components on the structural performance of the bridge. |
| Bridge loading           | • Bridge-specific assessment live loading models (BSALL) are derived from load measurement data: e.g., weigh-in-motion (WIM) data. This tends to be for major bridges rather than small and standard ones.  
  • Load testing may also be performed for some bridges. |
Table 3.5 Examples of most common and useful type of bridge monitoring installation in U.K. practice (in the order of most frequently mentioned to least frequently mentioned in the interviews).

<table>
<thead>
<tr>
<th>Area of interest</th>
<th>Examples and comments</th>
</tr>
</thead>
</table>
| Wind and flooding | 1. Wind speed and direction are monitored using anemometers. Wind speed is used as a parameter for a threshold check, particularly for long span bridges. Certain actions (e.g., bridge closure, traffic restriction, special investigation) are triggered when the wind speed is above a certain threshold value. However, threshold values are usually set based on historical experience and the maintenance manual rather than scientific reasoning.  
  2. Flood level is monitored and used as another parameter for a threshold check where certain actions are triggered (e.g., bridge closure, scour assessment, assessment of impact of debris or water pressure uplift) when the flood level is above certain threshold value. |
| Bearing and joint movement | This is to check whether the bearings or joints (e.g., expansion joints, saddles and anchorages of a suspension bridge) have their full range of movement as intended to accommodate the effects of variations in temperature and live load. Restricted movement indicates lock-up or seizure, which could have detrimental effects on the bridge structure. Temperature is often also monitored and correlated with bearing and joint movement data, which can then be used to investigate bearing and joint fixity (Webb et al., 2014). |
| Dynamic response | Monitoring of bridge dynamic response, such as global vibrations and bridge cable vibrations, has been used in some cases as a means of checking whether sufficient damping is in place to reduce fatigue problems and ensure no exceedance of the serviceability limit state (SLS) (e.g., no excessive vibrations), and thus inform whether extra damping is needed. |
| Wire breaks | Acoustic emission (AE) sensors have been used in certain limited cases to detect the number of wire breaks in prestressing tendons or suspension bridge tendons. The data is used to perform a threshold check for maximum permissible number of wire breaks to maintain structural integrity and sustain traffic loading. The key challenge is to understand the pre-existing condition (i.e., number of wires left) before the monitoring system is installed. |
| Bridge loading | Weigh-in-motion sensors are used in some cases to monitor traffic loading for a threshold check (i.e., detecting over-weight vehicles) and for bridge assessment purposes (e.g., generation of a realistic live load model). |
| Others | Other useful monitoring activities mentioned by some interviewees include: tell-tales for monitoring crack width, extensometers for monitoring foundation movement, CCTV cameras for traffic monitoring, strain gauges at fatigue critical locations for assessing fatigue risks, and corrosion sensors for measuring corrosion status. |
a structural issue has been raised, typically regarding concerns of bridge capacity deficiency due to either damage and deterioration or increased bridge loading. In certain limited cases mentioned by some interviewees, an FE model may also be used to investigate more detailed stress profiles (e.g., stress fields at critical connections), analyse complex structural behaviour (e.g., torsional effects, live load distribution, soil-structure interaction) or assess the effects of key strengthening actions. Bridge FE models are typically not kept and maintained by an asset owner on a permanent basis (refer to section 3.5.1 for more explanations), unless the bridge is a landmark structure of strategic importance (e.g., the Forth Road Bridge, the Clifton Suspension bridge). No example has been noted where FE models are kept and then used proactively to detect new problems (e.g., new damage).

### 3.4 Industry needs

#### 3.4.1 Key structural components or issues that “keep bridge practitioners awake at night”

Four overarching root causes that “keep bridge practitioners awake at night” have been identified based on the majority of interviewees’ responses. These are presented and explained as follows:

1. **The bridge component or issue is safety critical.**
   Safety critical issues can be considered from two perspectives: (a) structural integrity of the bridge; and (b) safety of people within the vicinity of the bridge (e.g., general public, inspectors or labourers on site). The former includes bridge scour, corrosion of concrete reinforcement or prestressing tendons, bearing and joint seizure, and bridge strike. The latter includes concrete spalling and insufficient load bearing capacity of bridge parapets.

2. **The bridge component or issue is difficult to inspect.**
   This is commonly referred to as “hidden defects” in the U.K., which includes two types of defects: (a) those which are difficult to access; and (b) those which are difficult to detect visually, even though they may be easy to access. The former includes any defects inside box girders (e.g., fatigue cracks, section loss due to corrosion), bridge scour, corrosion of concrete reinforcement or prestressing tendons, and half-joint defects. The latter includes fatigue cracks in welded sections.

3. **The bridge component or issue is difficult to manage.**
   Water management related issues, such as joint leakage, were highlighted by the
majority of interviewees as a key challenge in bridge O&M for two reasons: (a) it is the primary source of material degradation and structural deterioration (e.g., concrete corrosion, steel corrosion, bearing and joint seizure); and (b) waterproofing measures have often failed to perform as specified due to improper manufacturing and installation (e.g., bad detailing) or poor management and maintenance (e.g., application of de-icing salts).

4. **There is a large degree of uncertainty in ascertaining the actual behaviour related to the structural component or issue, which may result in the risk of sudden and unexpected failure modes.**

   This is often due to limited forewarning of certain structural failure modes or insufficient engineering understanding of how certain parts of the structure behave. The former includes sudden or brittle failure modes such as bucking and shear. The latter includes bridge scour, unexpected expansion joint failure, and half joint and hinge behaviour.

   The full list of critical bridge components and structural issues identified in the interviews are summarised in Table 3.6.
Table 3.6 Critical bridge components and structural issues identified in U.K. bridge O&M activities based on the industry interviews.

<table>
<thead>
<tr>
<th>Description</th>
<th>Examples and comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material degradation</td>
<td>1. <strong>Corrosion of reinforcement bars or prestressing tendons embedded within concrete</strong>&lt;br&gt;• Currently very difficult to detect and quantify satisfactorily.&lt;br&gt;• No satisfactory remedial measures to treat concrete bridge corrosion in current practice.</td>
</tr>
<tr>
<td></td>
<td>2. <strong>Determination of yield strength of reinforcement bars in concrete bridges</strong>&lt;br&gt;• Directly related to structural integrity.&lt;br&gt;• Existing non-destructive testing (NDT) techniques are limited in their accuracy and reliability in estimating this property.</td>
</tr>
<tr>
<td></td>
<td>3. <strong>Fatigue prone steel structures</strong>&lt;br&gt;• The internal condition of steel box type structures is difficult to inspect.&lt;br&gt;• The length of weld to inspect on large bridges is significant.</td>
</tr>
<tr>
<td>Joints and bearings</td>
<td>Joints and bearings are directly influenced by water management. It has been found that they often fail unexpectedly and well before the specified design service life in practice.</td>
</tr>
<tr>
<td></td>
<td>1. <strong>Half-joints (for some bridges)</strong>&lt;br&gt;• Critical to structural integrity.&lt;br&gt;• Difficult to access and inspect.</td>
</tr>
<tr>
<td></td>
<td>2. <strong>Expansion joints</strong>&lt;br&gt;• Often fail unexpectedly and prematurely.&lt;br&gt;• Their failure can induce build-up of local stresses and bending moments.</td>
</tr>
<tr>
<td></td>
<td>3. <strong>Bearing seizure</strong>&lt;br&gt;• Reduce the capacity for accommodating temperature and traffic load variations, and thus accelerate the failure of bearing components.&lt;br&gt;• Could lead to structural failure if the adjacent components are not originally designed for the induced local stresses and bending moments.</td>
</tr>
</tbody>
</table>

Continued on next page
3.4 Industry needs

Table 3.6 – continued from previous page

<table>
<thead>
<tr>
<th>Description</th>
<th>Examples and comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge scour</td>
<td>Scour is one of the most common causes of bridge failure worldwide (Wardhana and Hadipriono, 2003).</td>
</tr>
<tr>
<td></td>
<td>1. Bridge scour</td>
</tr>
<tr>
<td></td>
<td>• Currently difficult and sometimes dangerous to inspect for scour.</td>
</tr>
<tr>
<td></td>
<td>• There is often limited forewarning of impending failure and the consequence can be catastrophic (e.g. bridge collapse).</td>
</tr>
<tr>
<td>Other key issues</td>
<td>1. Concrete spalling</td>
</tr>
<tr>
<td></td>
<td>• Poses safety threats to people and live traffic underneath the bridge.</td>
</tr>
<tr>
<td></td>
<td>2. Bridge strike</td>
</tr>
<tr>
<td></td>
<td>• Poses immediate concern to structural integrity.</td>
</tr>
<tr>
<td></td>
<td>• Difficult to assess quickly and satisfactorily the structural condition after a bridge strike.</td>
</tr>
</tbody>
</table>

3.4.2 Key capability gaps in bridge condition appraisal

Overall, based on all interviewees’ responses, bridge owners and operators in the U.K. are mostly concerned about four key questions:

1. Is the bridge safe? (i.e., margin of safety)
2. How long will the bridge or bridge component remain safe? (i.e., remaining service life)
3. What is happening with the bridge? (i.e., real structural behaviour and structural performance)
4. When and how to intervene? (i.e., optimal maintenance routines)

Based on the literature survey (section 2.2) and all interviewees’ responses, five categories of capabilities in bridge condition appraisal were derived, which can be useful to bridge O&M. These are:

1. Damage detection
2. Damage criticality evaluation
3. Reserve load capacity assessment
4. Remaining service life prediction

5. “What-if” scenarios simulation

These are summarised and described in more detail in Table 3.7. It should be noted that while these are some common areas of interest, the specific capabilities required often depend heavily on the individual bridge structures and specific cases.

Table 3.7 Key capability gaps in bridge condition appraisal in the U.K.

<table>
<thead>
<tr>
<th>Capability type</th>
<th>Comments and examples</th>
</tr>
</thead>
</table>
| Damage detection         | Some interviewees were not particularly interested in developing new capabilities for damage detection itself but were more interested in how damage affects capacity. Others were interested in targeted damage detection where there are specific issues with inspection. Specifically:  
• Early warning and early detection of damage, particularly hidden defects.  
• Real time detection of critical damage, particularly when the bridges are at remote sites and the bridge owner/operator has a large portfolio of bridge assets to manage.  
In terms of specific damage types, these are summarised in section 3.4.1. |
| Damage criticality evaluation | Evaluation of damage criticality was of great interest to many interviewees as it directly informs which damage should be intervened first from a large list of damage recorded. Examples include:  
• Which bearings should be replaced first when they may have similar appearance from the outside?  
• Which cracks should be refurbished first when there are numerous cracks on a bridge?  
• Which bridge components or details are most critical from a fatigue sensitivity point of view?  
• How to measure concrete durability of a bridge in a non-destructive manner?  
A key challenge for many bridge practitioners is to identify the critical parts and the critical damage of their bridge structures. |
### Table 3.7 – continued from previous page

<table>
<thead>
<tr>
<th>Capability type</th>
<th>Comments and examples</th>
</tr>
</thead>
</table>
| Reserve load capacity estimation    | The primary concern for all interviewees was structural safety, which depends directly on both bridge loading and load capacity. Load testing is sometimes used in practice to evaluate load capacity.  
  - Some interviewees raised the issue that a stronger link between condition (e.g., damage and deterioration) and capacity needs to be established, as currently it is difficult to understand exactly how condition affects capacity. |
| Remaining service life prediction   | Remaining service life was another key area of interest to the interviewees as it is particularly useful for optimising maintenance and refurbishment routines (specifically, “when does a bridge component reach a state when intervention is needed and what sort of intervention is needed?”). For example, many interviewees raised the issue that some bridge components, especially bearings and joints, tend to fail well before their specified design life and often in an unexpected manner. Commonly raised examples include:  
  - Durability model for sliding materials such as bearings.  
  - Propagation of cracks over time.  
  - Durability model for concrete. |
| “What-if” scenarios simulation      | Some useful “what-if” scenarios identified by the interviewees include:  
  - Change of loading: e.g., additional traffic loading  
  - Extreme events: e.g., extreme winds, successive extreme heat.  
  - Hypothetical damage scenarios: e.g., bearing seizure, bridge strike, concrete corrosion. |
3.5 Industry views on bridge monitoring, modelling and model updating

3.5.1 Barriers and incentives to using bridge monitoring and modelling in practice

All interviewees were familiar with the concepts of SHM and FE modelling, and therefore they were able to provide their thoughts and comments on the use of bridge monitoring and modelling for O&M purposes.

Bridge monitoring

There are two types of monitoring. One is reactive monitoring for the purposes of further investigation and examination after specific issues have been identified by other means such as visual inspection. Most bridge monitoring activities in practice fall under this category. The other is proactive monitoring to detect anomalous behaviour or structural damage in near real time and therefore to enable more proactive or preventative maintenance.

The most highlighted and frequently mentioned barriers to using bridge monitoring (i.e., the views shared by the majority of interviewees) are summarised as follows:

1. Cost
   
   Budgets are limited for bridge O&M. Most of the budget is currently taken by condition improvement measures such as repair and replacement (e.g., bearing and joint replacement, concrete repair, strengthening against impact) to ensure structural safety and extend service life. Compared with physical repair, since bridge SHM does not directly improve bridge condition and its benefits are often unclear, it is often difficult to justify its deployment, particularly when the budget is tight. In addition to the cost of the bridge SHM system, there are also ongoing costs of maintaining the installed SHM system and employing consultants to perform data post-processing and interpretation. Another issue related to cost is the financing model, specifically, who should be paying for the bridge SHM system.

2. Value case for monitoring: reactive and targeted monitoring vs. proactive and untargeted monitoring
   
   Currently there is a dilemma faced when deciding between reactive monitoring and proactive monitoring.
3.5 Industry views on bridge monitoring, modelling and model updating

(a) The issue with proactive monitoring is that it is difficult to envisage what could go wrong with a bridge structure as there are a large number of potential issues that might arise during its service life. It is also challenging to identify at the start of a bridge’s service life where the critical and vulnerable parts of the bridge are, often due to insufficient knowledge of real structural behaviour and operating conditions. In addition, it is very difficult to address the cost-benefit of untargeted monitoring where a large number of sensors may be needed (with some built-in redundancies to account for sensor failures), as the end objectives and benefits are often less clearly defined. Two main questions raised by the interviewees were: (1) which bridge(s) and what part(s) of a bridge should be monitored when there is a large portfolio of bridge assets to manage; and (2) most bridge assets may not have any issues for a long period of time (e.g., 30 to 50 years) from the start of their service life, in which case what will the monitoring data be used for.

(b) On the other hand, there are two main issues with reactive monitoring: (1) the structural issue (e.g., damage) needs to be picked up first by other means such as visual inspection, and (2) it is difficult to determine the pre-existing condition of the bridge or bridge component, as sensors often measure changes of state rather than the absolute state (e.g., strain, displacement, number of wire breaks).

Currently, it is much easier to establish the value case for reactive and targeted monitoring in practice as it directly addresses the specific issues of concern, particularly for existing bridges.

3. Processing of SHM data

There are two overarching data challenges for bridge SHM: (1) How to extract useful information from bridge SHM data? (2) How to manage and process large and heterogeneous bridge SHM datasets? Most interviewees raised the issue that bridge monitoring data has often not been exploited satisfactorily due to the above-mentioned two challenges. Not much SHM data collected has been directly useful to bridge O&M. More often it is a case of “measuring things just for the sake of it”. In addition, there are many challenges for data processing such as data cleansing and data de-trending (i.e., removal of environmental trends in SHM data); and there is generally a lack of “sense making” and engineering interpretation of SHM data to explain the underlying structural behaviour.

4. Reliability and futureproofing of bridge SHM system

The most commonly raised practical issue is the reliability of the SHM system. Data quality has been found to be a common problem (e.g., due to cabling, power supply,
sensor failure). False positives are not uncommon (e.g., false detection of wire breaks, false detection of over-weight vehicles). More significantly, the lifetime of sensors and sensor systems is often much shorter than that of a bridge. SHM systems have often been found to deteriorate and fail more quickly than the monitored bridges, particularly for long term monitoring. Other practical issues include adaptability to future computer systems and data management platforms as well as who should manage and maintain the bridge SHM system.

Due to insufficient knowledge and appreciation of the benefits, it was difficult for the interviewees to come up with clear incentives for using bridge SHM systems as part of their bridge management processes. Most interviewees commented that they would consider using bridge monitoring if the above-mentioned barriers could be properly addressed. A few valid incentives were raised by some interviewees and these are summarised as follows:

1. **Cost reduction by reducing risks and uncertainties**
   One common question raised by the interviewees was: Can a bridge SHM system enable more targeted and meaningful spending on maintenance and refurbishment? In other words, “spend the right amount of money in the right place at the right time”. For example, it is costly and sometimes physically impossible to replace all bridge bearings, and many bearings have similar appearances from the outside even though some may have deteriorated and could cause detrimental effects to the bridge. One potential use case of bridge SHM data is to provide evidence regarding which bearings should be replaced.

2. **Better knowledge of real structural behaviour**
   Many interviewees mentioned that it would be good to have better insight and engineering understanding of the real structural behaviour, such as load path and load distribution, which could be used to analyse the effect of any observed damage on structural integrity and structural capacity.

3. **Remote management of bridges**
   Remote management is particularly useful when the bridge owner has a large portfolio of bridge assets to manage and maintain and these bridge assets are often difficult to access, i.e., at remote sites.

**Bridge modelling**

The most highlighted and frequently mentioned barriers to using bridge modelling for O&M purposes (i.e., the views shared by the majority of interviewees) are summarised as follows:
3.5 Industry views on bridge monitoring, modelling and model updating

1. **Model type**

   One key question raised by many interviewees is what type of structural analysis model should be used, especially if it were to be held by the bridge manager as part of the O&M datafiles for the bridge asset. Different use cases require different model fidelities. In addition, it may not be realistic in practice to model every detail and capture every damage scenario in a structural model.

2. **End benefits**

   Many interviewees raised the issue that FE modelling had rarely been needed so far and it was unclear to them why there is a need to keep an FE model with a bridge and for what purposes. The most common use case for an FE model has been when there is an increase in bridge loading and the model is then created for bridge assessment purposes.

3. **Practical issues**

   Three major practical issues were raised by the interviewees:

   (a) **Cost-benefit**: It is costly to model and analyse a large number of bridges and employ expensive consultants. It is also unclear who should keep the FE model for tens of years when the bridge remains in good condition and there appears to be no clearly defined use case.

   (b) **Liability**: There is a liability issue when using analysis models created by other people or organisations. In the U.K., if the owner keeps a model, it has an obligation to check the model to ensure there are no errors. The owner then needs to take legal responsibility for this model if anything goes wrong. Bridge owners in the U.K. tend to keep the drawings and technical approval documents but not the calculations and analysis models due to this liability issue.

   (c) **Software package**: FE software packages have evolved over the years. If an FE model is to be kept with the bridge asset, the issue of adaptation to new software packages and computer systems needs to be addressed. The alternative is to build an FE model from scratch every time it is needed.

   As for incentives to using FE modelling for bridge O&M, especially on a more frequent basis and if the model is to be kept with the bridge asset, it was generally very difficult for the majority of interviewees to come up with clear incentives due to insufficient knowledge and appreciation of its benefits and the above-mentioned barriers.
3.5 Industry views on bridge monitoring, modelling and model updating

3.5.2 Industry perspectives on bridge model updating

In current U.K. industry practice, the generation of a more realistic bridge analysis model is not achieved through solving an “inverse problem” by back calculating model parameters and modifying modelling assumptions based on sensor measurements of structural response. Rather, a direct approach is adopted by gathering as much information as possible about the physical properties of the bridge, typically through condition surveys (refer to section 3.3.1 regarding Level III assessment).

The general research approach to bridge model updating, which is by solving an “inverse problem”, and the common research goal of performing damage detection, which is through detecting a local reduction in structural stiffness, were described to each interviewee. Only eight out of the nineteen interviewees had heard of the research approach before the interview (They are C2, C11, C12, C13, C14, C15, C16, C18 – refer to Table 3.1). Unlike other parts of the interview where the majority or the most common views are distilled and presented, the purpose for this part of the interview is to gather valid comments and questions raised by the expert bridge practitioners, especially those who have extensive experience in bridge modelling and have familiarity with the bridge model updating concept. The gathered industry perspectives on bridge model updating are summarised as follows:

1. On bridge model updating research methodologies

   (a) One commonly raised issue is reliability. Specifically, there seems to be a lack of further verification and validation as well as additional engineering interpretation and evidence if the model updating results were to be fully relied on in practice. Some interviewees (C11, C12, C13, C18) raised the issue that in general, it is easy to justify the measurements by adjusting model parameters but difficult to make predictions as past predictions have often been found to be incorrect or unreliable.

   (b) In addition, the model updating research approach of solving an inverse problem to detect structural damage is currently outside the framework of what most engineers would operate in terms of signing off the capacity of a bridge structure. To some interviewees (C1, C2, C13, C14, C18), it also seems to involve much more work and effort compared with the existing industry approach of demonstrating that a bridge is safe and performing satisfactorily.

2. On bridge model updating research outputs

   One of the main goals of bridge model updating in current research is to perform damage detection through detecting a local reduction in structural stiffness.
3.6 Disconnects between research and practice

(a) Regarding the performance of damage detection using the model updating approach (i.e., detecting a local stiffness reduction by solving an “inverse problem”), one key feedback raised by some expert bridge professionals (C2, C6, C11, C13 and C14) is that there is doubt on whether this approach can detect any actual damage of concern in a reliable and adoptable manner. Take corrosion of steel reinforcement bars as an example. This common type of damage mainly affects the available tensile strength of steel rather than stiffness of the section. If this approach is to detect early stages of corrosion (e.g., 5% loss of section), the effect of reinforcement corrosion on reduction in structural stiffness may be negligible and therefore the damage may not be detected. The level of sensitivity of the sensor data to structural damage was also cast into doubt by some interviewees. On the other hand, if this approach is to detect more severe concrete corrosion and section losses, these are likely to be detected first from visual signs (e.g., signs of rust staining on the soffit of the structure) before any detectable change from bridge SHM and model updating occurs, so visual inspection may be a much more cost-effective method in this scenario based on the practitioners’ views.

(b) In addition, bridge modelling in current practice is largely, if not solely, driven by capacity assessment rather than damage detection. The majority of interviewees are more interested in the actual capacity of their bridge assets and how structural damage affects bridge capacity, rather than damage detection alone.

(c) Other areas of interest mentioned by some interviewees include: (i) better understanding of real structural behaviour and the underlying causes of any structural damage or anomalous structural behaviour in order to evaluate the criticality of each damage to structural integrity and structural capacity (C4, C9, C14); (ii) the use of reduced safety factors or load models in bridge assessment (C2, C11, C16); and (iii) modifying boundary conditions to reflect support restraints more realistically (C2, C11).

3.6 Disconnects between research and practice

Based on the results of both the literature survey on bridge model updating (refer to section 2.2) and the industry interviews, the disconnects between research and practice were identified and examined under two categories:

1. Disconnects between research outputs from bridge model updating studies and industry needs in bridge O&M.
2. Disconnects between research methodologies of bridge model updating and the industry’s approach to bridge condition appraisal.

The disconnects between research outputs from bridge model updating studies and industry needs in bridge O&M are summarised as follows:

1. **On damage detection**

   One of the key end objectives in existing bridge model updating research is to perform damage detection and assessment through detection of local reduction in structural stiffness. By examining the outputs or results of bridge model updating research (refer to section 2.2.7) based on the expert practitioners’ views in this area, a number of key issues have been identified:

   (a) Existing bridge model updating research for damage detection often assumes that localised damage results in a local reduction in stiffness, which can then be detected from sufficient change of structural behaviour. This may not be the case for many types of bridge damage such as corrosion of reinforcement bars inside concrete, which mainly affects the tensile strength capacity of steel rather than elastic stiffness of the section under normal operating conditions (refer to section 3.5.2 for more details).

   (b) Based on the outputs or results from the surveyed research studies (refer to section 2.2.7), most bridge model updating research methodologies have not specified the exact types of bridge damage that can be detected and whether there are certain types of bridge damage that may not be detected. In particular, there is little research on addressing those specific damage concerns that “keep bridge practitioners awake at night” (refer to section 3.4.1 for more details). Therefore, it is difficult to evaluate the relative performance of damage detection by the model updating approach used in research compared with that by the current visual inspection approach used in industry. Key performance evaluation criteria of damage detection, as summarised based on the expert practitioners’ views (refer to section 3.4.2), include: detection accuracy and reliability, capability of early detection, capability of detecting hidden defects, cost-benefit analysis.

   (c) In addition, based on the industry interviews, many bridge practitioners are mainly interested in the underlying cause of any identified damage and the criticality of each damage to structural integrity, i.e., how structural damage affects structural capacity (refer to section 3.4.2 for more details). These issues have not been adequately addressed in existing bridge model updating research for damage
detection, which are of greater interest to many bridge practitioners than damage
detection alone as they are critical for optimising maintenance actions.

2. **On capacity assessment**

   Based on the industry interviews, bridge practitioners are mainly interested in the
   safety and margin of capacity of their bridge assets, which are directly related to the
   actual bridge loading and load capacity. Moreover, structural modelling of bridges in
   the current framework of the U.K. industry is driven by capacity assessment rather
   than damage detection (refer to section 3.3). A limited amount of research in bridge
   model updating has so far been focused on improving capacity assessment. Since
   bridge FE modelling for O&M purposes is costly and involves a great amount of effort
   in practice, it may be difficult to establish the value case for implementing bridge
   model updating purely for the purpose of damage detection. Key areas of interest for
   capacity assessment, based on the expert practitioners’ feedback (refer to sections 3.4.2
   and 3.5.2), include: (i) how structural damage affects load capacity; (ii) real structural
   behaviour such as load path and load distribution; (iii) boundary conditions; and (iv)
   the use of reduced safety factors and load models.

   Regarding the disconnects between research methodologies of bridge model updating and
   the industry’s approach to bridge condition appraisal, there are many practical issues involved
   when implementing the research methodologies of bridge model updating in practice. These
   are described in more detail in section 3.5.1. In summary, these include: the liability issue for
   bridge owners to keep FE models, adaptability to future upgrade of software packages and
   computer system, reliability and futureproofing of the installed SHM system, FE model and
   SHM system ownership, and cost-benefit analysis. In addition, FE modelling is rarely used in
   current bridge O&M practice and it is predominantly used in a reactive and one-off manner
   to address known and specific issues, while the academic vision of bridge “digital twinning”,
   which updates the model in near real time as new monitoring data becomes available, requires
   more frequent and proactive use of the analysis model in order to realise its value.

3.7 **Summary**

   This chapter has investigated the reasons behind the lack of industry uptake of bridge
   monitoring, modelling and model updating for bridge operation and maintenance (O&M)
   in the U.K. context. A series of semi-structured interviews were conducted with 19 expert
   bridge professionals in the U.K. to understand current industry practice and needs in bridge
   condition appraisal and to collect industry views on using monitoring, modelling and model
updating for bridge O&M in practice. The findings from these industry interviews were compared with those found from the literature survey in section 2.2 to identify the disconnects between research and practice. A summary of the key findings is listed below:

- **Disconnects between research outputs from bridge model updating and industry needs in bridge O&M**
  Most existing bridge model updating studies for damage detection purposes have not specified the exact types of damage that can be detected or focused on addressing specific damage concerns of the bridge practitioners (e.g., material degradation, joint seizure, bridge scour). The assumption that localised damage results in a local reduction in stiffness is subject to question by the practitioners, as many common types of bridge damage may not induce noticeable change in structural stiffness that existing model updating techniques would identify. In addition, compared with damage detection, many bridge practitioners are mainly interested in bridge capacity assessment and the understanding of real structural behaviour (e.g., load path, load distribution) to evaluate the criticality of any observed damage on structural integrity and structural capacity.

- **Disconnects between research methodologies of bridge model updating and the industry’s approach to bridge condition appraisal**
  Bridge model updating is outside the current framework in which bridge practitioners operate to ensure a bridge structure is safe and has sufficient capacity. Structural modelling for bridge O&M in practice is driven by capacity assessment rather than damage detection, and it is predominantly a one-off exercise rather than routine practice. There are also many practical issues for implementing the bridge model updating research methodologies in practice, including cost of monitoring and modelling, the liability issue for bridge owners to keep FE models, and adaptability to future upgrade of software packages and computer system.
Chapter 4

Chebsey Bridge Monitoring Programme

4.1 Introduction

Based on the findings from the literature review and the industry interviews, and in particular, the disconnects between research and practice regarding bridge monitoring and model updating, how monitoring data may be used to improve structural assessment of bridges through a better understanding of in-service structural behaviour and structural utilisation (e.g., “margin of capacity”) was investigated. An instrumented operational railway bridge, known as the Chebsey bridge, was used as the case study.

This chapter presents the monitoring programme for the Chebsey bridge, which has been instrumented with a dense network of fibre optic sensors (FOS) since its construction in 2015. Specifically, the design and construction information of the bridge, the monitoring system, and the data collected are presented. The processing and interpretation of the monitoring datasets are presented over the following chapters (Chapters 5 to 8).

4.2 The Chebsey bridge

The Chebsey bridge, also referred to as Underbridge 11 (UB11), is located in Staffordshire, U.K. It carries one rail line across the West Coast Main Line. The bridge was constructed in 2015 as part of the Staffordshire Area Improvement Programme (SAIP) (VolkerRail, 2017). It is now managed by Network Rail.

The Chebsey bridge is an 11.2 m span prestressed concrete girder bridge, as shown in Figure 4.1. The bridge deck consists of nine 11.9 m precast prestressed concrete girders (two edge TYE7 beams and seven internal TY7 beams (Banagher Precast Concrete, 2014)) as well as a cast in-situ reinforced concrete infill deck slab, as shown in Figure 4.2a. The nine
4.3 Monitoring system

longitudinal girders were pre-manufactured at the Laing O’Rourke Explore Industrial Park (EIP) manufacturing facility in Worksop, U.K. and then transported to the construction site for installation. The girders are supported on elastomeric bearings, as shown in Figure 4.1c and Figure 4.2b. The concrete deck slab is reinforced with steel reinforcing bars, and the nine girders are tied together using transverse steel cross ties, as shown in Figure 4.2a. The rail line, as shown in Figure 4.1b, is supported on ballasted track (350 mm depth ballast) with a cant of 100 mm as the rail line is slightly curved.

The main prestressed concrete girders of the bridge deck were designed according to the provisions of EN 1992 – Eurocode 2: Design of concrete structures (CEN/TC250, 2005). Stress profiles across the bridge deck under different load combinations at different stages of construction and operation were checked against the corresponding serviceability stress limits. The design was governed by this serviceability limit state (SLS) check. As for the design train load, the precast girders were designed to sustain the live load actions of Load Model 71 (LM71), which is the standard load model for normal rail traffic, as provided in part 2 of Eurocode 1 and shown in Figure 4.3 (CEN/TC250, 2003). It should be noted that LM71 does not represent an actual train, but rather, it is intended to produce an envelope of load effects (bending moments and shears) that would be caused by normal rail traffic.

4.3 Monitoring system

The bridge was instrumented with a dense network of fibre optic sensors (FOS) during its construction in 2015, with data collection exercises conducted at multiple stages during its construction and operation. The FOS technologies deployed are capable of measuring strain and temperature changes. Two types of FOS were installed on the bridge. One was a discrete or point-based FOS using Fibre Bragg gratings (FBGs), which can measure dynamic strain and temperature at specific point locations. The other was a distributed FOS based on Brillouin Optical Time Domain Reflectometry (BOTDR), which can measure static strain and temperature changes which are averaged over a certain gauge length (e.g., every 50 cm) and then taken at a fixed distance apart along the length of a fibre optic cable. To decouple the effects of temperature change and the change of physical strain on FOS wavelength measurements, temperature was measured using separate fibre optic sensors for the purpose of temperature compensation when converting FOS wavelength measurements to strain data.
Fig. 4.1 Images of the Chebsey bridge (Images by Liam J. Butler): (a) completed bridge; (b) top of the bridge; and (c) bottom of the bridge.
Fig. 4.2 Selected bridge drawings (Courtesy of Network Rail): (a) transverse cross-section of the bridge deck (with nine girders labelled); and (b) longitudinal elevation of the bridge deck showing support details at the abutment (with elastomeric bearings and concrete infill).
4.3 Monitoring system

Fig. 4.3 Eurocode 1 Load Model 71 (LM71) with characteristic values ($Q_{vk}$ represents the point load component and $q_{vk}$ represents the distributed load component) (Courtesy of Liam J. Butler).

For FBG sensors, the mechanical strain (considering temperature compensation), $\Delta \varepsilon_m$, can be calculated using Equation 4.1:

$$\Delta \varepsilon_m = \frac{1}{k_e} \left[ \left( \frac{\Delta \lambda}{\lambda_0} \right)_S - \frac{k_T \left( \frac{\Delta \lambda}{\lambda_0} \right)_T}{k_{T_T}} \right] - \alpha_{sub} \frac{\Delta \lambda}{\lambda_0} T_T$$  \hspace{1cm} (4.1)

where

- $\left( \frac{\Delta \lambda}{\lambda_0} \right)_S$ = relative wavelength shift of strain-measuring FBG sensor
- $\left( \frac{\Delta \lambda}{\lambda_0} \right)_T$ = relative wavelength shift of temperature compensating FBG sensor
- $k_e$ = gauge factor for FBG sensor (typical value = 0.78)
- $k_T$ = experimentally derived constant for temperature compensating FBG sensor (typical value = $7 \times 10^{-6}$)
- $k_{T_T}$ = change in the refractive index of glass (typical value = $1.12 \times 10^{-5}$)
- $\alpha_{sub}$ = linear coefficient of thermal expansion of the substrate material (= $10 \times 10^{-6}/{°C}$ for concrete)

For live load response, i.e., dynamic strain under train-passage events, it was assumed that the change in ambient temperature during each train-passage event was negligible as the time window for a train-passage event was very short (typically a few seconds). Under such circumstances, Equation 4.1 may be simplified to Equation 4.2 for dynamic strain:

$$\Delta \varepsilon_m = \frac{1}{k_e} \left( \frac{\Delta \lambda}{\lambda_0} \right)$$  \hspace{1cm} (4.2)
For BOTDR, the mechanically induced strain from Brillouin frequency shift can be calculated using Equation 4.3:

\[ \nu_b(s) = \nu_{b0} + M \varepsilon(s) \] (4.3)

where

\( \nu_b(s) \) = Brillouin frequency shift as a function of distance

\( \nu_{b0} \) = Brillouin frequency shift with zero induced strain

\( M \) = a constant of proportionality

\( \varepsilon(s) \) = thermally and mechanically induced strain as a function of distance

In terms of sensor resolution, the strain-measuring FBGs can measure strain changes within \( \pm 5 \mu \varepsilon \) and the temperature-measuring FBGs can measure temperature changes within \( \pm 1.0^\circ C \). The BOTDR cables used were capable of measuring static distributed strain within \( \pm 50 \mu \varepsilon \) and the temperature-measuring BOTDR can measure temperature changes within \( \pm 1.0^\circ C \) (Butler et al., 2016). Cable coatings for both FBG and BOTDR sensors were carefully selected to ensure long-term sensor robustness and durability. Further details of the sensor specification and installation were reported in Butler et al. (2016).

Figure 4.4 provides an overview of the monitoring system for the Chebsey bridge. The sensor arrangement was designed to measure the time-dependent strain evolution (e.g., prestress loss) as well as the load distribution, and in particular, the longitudinal and transverse bending behaviour, of the bridge deck. Because the bridge deck is symmetrical and the rail line is at the centreline of the bridge deck, half of the deck, beams BM1 to BM5, were instrumented. The other edge beam BM9 was also instrumented for checking symmetry in structural behaviour.

Figure 4.5 provides further details of the sensor arrangement. For each beam, longitudinal strain was measured along one top and one bottom prestressing strand. There are 20 strain-measuring FBGs per sensor array spaced at one metre centre to centre and 6 temperature-measuring FBGs per sensor array (one sensor array for each instrumented beam). BOTDR cables were also installed alongside the FBGs. The fibre optic sensor arrangement for the instrumented beams is shown in Figure 4.6. These sensors were installed during the manufacturing of the beams. In addition, transverse strain was measured along one top reinforcement bar within the in-situ concrete deck slab and along one transverse steel reinforcement cross tie, as shown in Figure 4.5b, at both the mid-span and the east quarter span locations.
Fig. 4.4 The Chebsey bridge monitoring system overview.
4.3 Monitoring system

Fig. 4.5 The Chebsey bridge monitoring system details: (a) side longitudinal elevation along the bridge deck (with longitudinal sensor locations labelled L1 to L10 with 1000 mm spacing); and (b) transverse cross-section of the bridge deck (with instrumented beams labelled BM1 to BM5 and BM9).
Fig. 4.6 Fibre optic sensor arrangement along prestressed concrete beams: (a) internal beams (BM2 to BM5); and (b) edge beams (BM1 and BM9).
4.4 Data collection

As mentioned previously, the bridge has been monitored intermittently since its construction in 2015. Both FBG and BOTDR data were collected at multiple stages during the bridge’s construction and operation (e.g., beam casting, beam installation, concrete deck casting, train-passage events), as summarised in Table 4.1. As mentioned in section 4.3, the monitoring system was mainly designed to measure time-dependent strain evolution (e.g., prestress loss) and load distribution of the bridge deck.

<table>
<thead>
<tr>
<th>Date</th>
<th>No. days</th>
<th>Description</th>
<th>Data collection</th>
</tr>
</thead>
<tbody>
<tr>
<td>9 Jan 2015</td>
<td>0</td>
<td>Concrete casting of TYE7 beams (edge beams)</td>
<td>BM1 and BM9</td>
</tr>
<tr>
<td>9–13 Jan 2015</td>
<td>0–4</td>
<td>Initial curing prior to the transfer of prestress for TYE7 beams</td>
<td>BM1 and BM9 - continuous measurement</td>
</tr>
<tr>
<td>13 Jan 2015</td>
<td>4</td>
<td>Pre- and post-transfer of prestress for TYE7 beams</td>
<td>BM1 and BM9</td>
</tr>
<tr>
<td>22 Jan 2015</td>
<td>13</td>
<td>Concrete casting of TY7 beams (internal beams)</td>
<td>BM2 and BM3</td>
</tr>
<tr>
<td>22–29 Jan 2015</td>
<td>13–20</td>
<td>Initial curing prior to the transfer of prestress for TY7 beams</td>
<td>BM2 and BM3 - continuous measurement</td>
</tr>
<tr>
<td>29 Jan 2015</td>
<td>20</td>
<td>Pre- and post-transfer of prestress for TY7 beams</td>
<td>BM2 and BM3</td>
</tr>
<tr>
<td>5 Mar 2015</td>
<td>55</td>
<td>Second stage concrete casting on TYE7 beams (as part of the finished bridge deck slab)</td>
<td>BM1 and BM9</td>
</tr>
<tr>
<td>14 Apr 2015</td>
<td>95</td>
<td>Outdoor storage in precast facility (early-age curing for 3 months)</td>
<td>All instrumented beams (BM1 to BM5, BM9)</td>
</tr>
</tbody>
</table>

Continued on next page
### Table 4.1 – continued from previous page

<table>
<thead>
<tr>
<th>Date</th>
<th>No. days*</th>
<th>Description</th>
<th>Data collection</th>
</tr>
</thead>
<tbody>
<tr>
<td>6 July 2015</td>
<td>178</td>
<td>Prior to casting of infill concrete deck; after transportation of beams to site, installation of beams on the bridge abutments (both on 30 June 2015) and installation of reinforcing steel</td>
<td>BM3 and BM9</td>
</tr>
<tr>
<td>13 July 2015</td>
<td>185</td>
<td>Pre- and post-casting of infill concrete deck</td>
<td>BM1, BM2, top reinforcing steel in deck slab, transverse steel cross ties</td>
</tr>
<tr>
<td>21 July 2015</td>
<td>193</td>
<td>Initial curing of infill concrete deck</td>
<td>All instrumented components (BM1 to BM5, BM9, top reinforcing steel in deck slab, transverse steel cross ties)</td>
</tr>
<tr>
<td>28 Nov 2015</td>
<td>292</td>
<td>Temporary haul road construction (with added live load)</td>
<td>All instrumented components</td>
</tr>
<tr>
<td>14 Mar 2016</td>
<td>430</td>
<td>Prior to service</td>
<td>All instrumented components</td>
</tr>
<tr>
<td>14 July 2016</td>
<td>552</td>
<td>4 months in-service</td>
<td>All instrumented components - data collected for strain baseline and 4 train-passage events (1 passenger train, 1 freight train and 1 single diesel engine)</td>
</tr>
<tr>
<td>7 July 2017</td>
<td>910</td>
<td>1 year and 4 months in-service</td>
<td>All instrumented components - data collected for strain baseline and 2 train-passage events (1 passenger train and 1 freight train)</td>
</tr>
<tr>
<td>27 Sept 2017</td>
<td>992</td>
<td>1 year and 6.5 months in-service</td>
<td>All instrumented components - data collected for strain baseline and 6 train-passage events (2 passenger trains and 4 freight trains)</td>
</tr>
</tbody>
</table>

* Note: This refers to number of days after casting of TYE7 beams. Note that TY7 beams and TYE7 beams were cast on different dates.
Chapter 5

Structural Behaviour Part I – Prestress Loss Behaviour

5.1 Introduction

This chapter presents the evaluation of one important aspect of the structural behaviour of the Chebsey bridge: prestress loss, which is a critical parameter in structural modelling and capacity assessment of prestressed concrete bridges. Specifically, the study evaluates the prestress losses of four instrumented prestressed concrete beams of the Chebsey bridge, which are two TY7 internal beams (BM2 and BM3) and two TYE7 edge beams (BM1 and BM9), for the first two and a half years since the beams were cast.

Prestressed concrete bridges comprise a significant proportion of the bridge stock both in the U.K. and around the world. For example, the U.K.’s rail sector alone has more than 25,000 bridges and around one tenth of them are prestressed concrete bridges (Network Rail, 2015). In addition, with the introduction of standard precast prestressed concrete beams for bridge construction as well as growing industry familiarity and technical competency in prestressed concrete construction, prestressed concrete bridges are likely to be an increasingly competitive and cost-effective option for future bridge construction (Bourne, 2013). Therefore, it is essential that prestressed concrete bridges are managed and maintained properly to ensure the safety and serviceability of these infrastructure assets in the future.

The level of prestress in the beams in prestressed concrete bridges is a critical performance indicator because it governs both structural safety (e.g., remaining load capacity) and serviceability (e.g., concrete cracking, in-service deflections). The magnitude of prestressing force decreases gradually throughout the life of a prestressed concrete bridge. Both the magnitude and the rate of prestress loss are affected by many interdependent factors such as
5.1 Introduction

time-dependent material properties (e.g., concrete modulus of elasticity, concrete strength), material behaviour (e.g., concrete shrinkage and creep, steel relaxation) and structural conditions (e.g., reinforcement type, anchorage conditions).

Current practice for evaluation of existing prestressed concrete bridges (and concrete bridges in general) depends primarily on visual inspection of the condition of the concrete and steel. Acoustic emission (AE) sensors are used in some limited cases to detect breaks in the prestressing wire (Middleton et al., 2016; Webb et al., 2014). Visual assessment poses issues of subjectivity, unreliability, late detection of damage and difficulty of relating observed damage to actual load capacity (Lea and Middleton, 2002; Moore et al., 2001). As for AE sensors, they have an inherent difficulty in assessing the absolute amount of damage rather than its rate of change and they may produce data containing false positives (Middleton et al., 2016).

An accurate understanding of short-term and long-term prestress losses by structural monitoring would be beneficial for assessing structural capacity and improving decision making for the management and maintenance of this type of bridge. Without monitoring data, conservative assumptions on the remaining level of prestress need to be made in bridge assessment, for example, based on design codes and standards and design guidance documents (American Association of State Highway and Transportation Officials [AASHTO], 1998; American Concrete Institute [ACI] Committee 209, 1997; CEN/TC250, 2004; Hendy and Smith, 2007; International Federation for Structural Concrete [fib], 1993; Naaman, 2004; O’Brien et al., 2012). Very few field studies have been conducted to measure prestress losses in bridges during their construction and operation. Consequently, the nature of the progression of prestress losses in real world bridge applications is largely unknown and any resulting data comparing actual losses versus code predictions has rarely been available.

This study serves as the culmination of a previous study Butler et al. (2016) on early-age behaviour of prestressed concrete beams prior to concrete deck casting. Prestress loss mechanisms were investigated in detail including immediate prestress losses due to elastic shortening of concrete and time-dependent prestress losses due to steel relaxation, concrete shrinkage and concrete creep. Prestress loss predictions were calculated using both European and American standards (EC2 and AASHTO), which were then compared with the measured prestress losses. Both simplified and advanced time-step methods were used to provide more refined loss predictions by taking into account the interrelationships between various prestress loss mechanisms and the total prestress force at the time of interest.
5.2 Code predictions of prestress loss

5.2.1 Prestress loss development of precast prestressed concrete beams

By considering the change in prestressing force, changes in material properties and changes in section geometry (different stages of construction) over time, the strains at various longitudinal locations along the beams and vertically through the cross-section depth can be calculated at any time using Equation 5.1:

\[
\varepsilon(x,y,t) = \frac{P(x,t)}{A_c(t)E_c(t)} + \frac{[M_{sw}(x,t) + M_{qp}(x,t) + P(x,t)e(t)]y}{E_c(t)I_c(t)}
\]  

(5.1)

where \(x\) is the longitudinal distance along the length of the beam; \(y\) is the vertical distance relative to the geometric centroid of the beam cross-section; \(t\) is time; \(P\) is the total prestressing force in the tendons; \(M_{sw}\) is the bending moment due to self-weight; \(M_{qp}\) is the bending moment due to quasi-permanent service load; \(e\) is the eccentricity of the centroid of all tendons relative to the centroid of the beam cross-section (un-cracked); \(A_c\) is the area of the beam cross-section; \(E_c\) is the concrete modulus of elasticity; and \(I_c\) is the second moment area of the beam cross-section.

In addition to instantaneous prestress losses due to elastic shortening, anchorage set and friction (friction loss only occurs in post-tensioned construction), the prestressing force also changes with time as various time-dependent material-related mechanisms evolve, such as steel tendon relaxation, concrete shrinkage and concrete creep. In conjunction, the various stages of construction and their associated quasi-permanent loading all influence the change in total prestressing force. To capture the evolution of this changing force, a time-step method may be used to calculate the sequential changes in prestressing force (Hendy and Smith, 2007; Naaman, 2004). As the level of prestress in steel tendons and various time-dependent prestress loss mechanisms (i.e., steel relaxation, concrete shrinkage and concrete creep) are interrelated, incremental prestress losses for these mechanisms as well as the remaining prestressing force need to be re-calculated at each time step (e.g., at each day or between successive construction stages). Furthermore, it may be significant to account for the effect of differential shrinkage that arises from the casting of concrete deck after the prestressed concrete beams have undergone some initial levels of relaxation, creep and shrinkage.

The accuracy of prestress loss predictions depends not only on the accuracy of code equations and provisions, but also on the accuracy of key input parameters. These parameters include material properties, section geometric properties, applied loadings, boundary conditions, environmental conditions, etc. Based on the bridge designer’s calculations, the initial prestressing force was assumed to be 209 kN for each prestressing tendon (corresponding to
a stress in each tendon of 1393 MPa). This value was later confirmed by the prestressing facility operator’s calculations. All of the prestressed concrete beams were cast using self-compacting concrete (SCC) of strength class C60/75. The material and section geometric properties of the beams are summarised in Table 5.1.

Prestress loss estimation is a complex problem, as it is influenced by a number of factors. These include:

1. Time to transfer of prestressing force (i.e., detensioning) since concrete beam casting
2. Age of girders at the time of deck casting
3. Type of prestressing strands
4. Concrete material properties (e.g., compressive strength, modulus of elasticity)
5. Initial stresses in concrete (e.g., due to prestress)
6. Storage conditions (e.g., relative humidity, temperature, amount of air exposure)

In general, (1) affects the concrete properties at the time of transfer and thus the amount of elastic shortening; (2) affects the amount of differential shrinkage; (3) affects steel properties (e.g., relaxation properties, maximum allowable prestress force), which in turn affect various prestress loss mechanisms (e.g., low relaxation steel has lower relaxation loss compared with stress relieved steel); (4) affects all time-dependent prestress loss mechanisms; (5) affects creep loss; and (6) affects various prestress loss mechanisms, particularly shrinkage loss. A sensitivity analysis has been performed to quantitatively examine the effect of each factor and thus determine their relative importance to the prestress loss predictions (refer to section 5.2.5).

To provide a basis from comparing prestress losses which have been calculated from the fibre optic sensor strain data, prestress losses were predicted using three different methods with varying levels of model fidelity. Two of these methods are based on Eurocode 2: one basic method and one time-step method (with time-steps of one day) (CEN/TC250, 2004; Hendy and Smith, 2007). The third method is based on AASHTO-LRFD, which is a time-step method specifically tailored for segmental construction (i.e., staged construction) of prestressed concrete girder bridges (with time-steps corresponding to different construction stages) (American Association of State Highway and Transportation Officials [AASHTO], 1998; Naaman, 2004). These three methods are described in the following three subsections (5.2.2 to 5.2.4).
5.2 Code predictions of prestress loss

Table 5.1 Material and geometric properties of prestressed concrete beams.

<table>
<thead>
<tr>
<th>Material or geometric property</th>
<th>TY7 beams</th>
<th>TYE7 beam</th>
</tr>
</thead>
<tbody>
<tr>
<td>Casting date</td>
<td>22 Jan 2015</td>
<td>9 Jan 2015</td>
</tr>
<tr>
<td><strong>Concrete material properties</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water/cementitious materials ratio</td>
<td>0.36</td>
<td>0.36</td>
</tr>
<tr>
<td>Slump flow</td>
<td>760 mm</td>
<td>770 mm</td>
</tr>
<tr>
<td>Design $f_{ck,cube}$ (28 days)</td>
<td>75 MPa</td>
<td>75 MPa</td>
</tr>
<tr>
<td>$f_{ck,cube}$ (28 days)</td>
<td>89.6 MPa</td>
<td>90.7 MPa</td>
</tr>
<tr>
<td>$f_{ck,cube}$ (7 days)</td>
<td>75.8 MPa</td>
<td>76.7 MPa</td>
</tr>
<tr>
<td>$f_{ck,cube}$ at transfer</td>
<td>60.5 MPa</td>
<td>65.8 MPa</td>
</tr>
<tr>
<td>Estimated $E_{cm}$ at transfer</td>
<td>35,300 MPa</td>
<td>36,210 MPa</td>
</tr>
<tr>
<td><strong>Steel material properties</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Prestressing steel (7 wire strand)</td>
<td>16 strands</td>
<td>19 strands</td>
</tr>
<tr>
<td>$f_{pu}$</td>
<td>1860 MPa</td>
<td>1860 MPa</td>
</tr>
<tr>
<td>$E_p$</td>
<td>195,000 MPa</td>
<td>195,000 MPa</td>
</tr>
<tr>
<td>$\rho_{1000}$ (elongation after 1000 h)</td>
<td>2.5%</td>
<td>2.5%</td>
</tr>
<tr>
<td><strong>Section geometry</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$A_{strand}$</td>
<td>150 mm$^2$</td>
<td>150 mm$^2$</td>
</tr>
<tr>
<td>Beam length, $L$</td>
<td>11.9 m</td>
<td>11.9 m</td>
</tr>
<tr>
<td>Area of uncracked gross section, $A_c$</td>
<td>272,194 mm$^2$</td>
<td>398,330 mm$^2$</td>
</tr>
<tr>
<td>Moment of inertia of uncracked gross section, $I_c$</td>
<td>$1.26 \times 10^{10}$ mm$^2$</td>
<td>$1.75 \times 10^{10}$ mm$^2$</td>
</tr>
<tr>
<td>Eccentricity of tendons in uncracked gross section, $e_{cp}$</td>
<td>80.9 mm</td>
<td>125 mm</td>
</tr>
</tbody>
</table>

1 Measured values are based on cube specimens cured under controlled conditions, i.e. in water bath.
2 Estimate based on measured maturity versus temperature data provided by contractor (detensioning of TYE7 beams occurred 94 h after casting).
3 $E_{cm(1)} = (f_{cm}(1)/f_{cm}(28))^{0.3} E_{cm(28)}$; $E_{cm(28)} = 22(f_{cm}(28)/10)^{0.3}$; $f_{ck(t)} = 0.8 f_{ck,cube(t)}$.
4 Based on supplier specification sheets.
5 Based on designers’ calculation sheets. Value relative to centroid of the beam cross section (below the centroid).
5.2 Code predictions of prestress loss

5.2.2 Prestress loss calculation: EC2 basic method

Table 5.2 presents a summary of EC2 equations for estimating prestress losses. In this study, due to the detensioning procedures adopted by the contractor at the precast facility, it was assumed that any immediate losses due to anchorage set were negligible. Losses due to steel relaxation, elastic shortening of concrete, concrete shrinkage and concrete creep were considered. Prestress losses in the prestressed concrete beams were evaluated at a series of monitoring and construction stages, as previously shown in Table 4.1 of section 4.4.

Table 5.2 Summary of Eurocode 2 formulae for prestress loss predictions.

<table>
<thead>
<tr>
<th>Prestress loss mechanism</th>
<th>Eurocode 2 formulae</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation of tendons</td>
<td>$\Delta P_{REL,i} = 6.6A_p\sigma_{pi}\rho_{1000}e^{9.1\mu}\left(\frac{t}{1000}\right)^{0.75(1-\mu)} \times 10^{-6}$ where $\mu = \frac{\sigma_{pi}}{\sigma_{p_0}}$</td>
</tr>
<tr>
<td>Elastic shortening of concrete</td>
<td>$\Delta P_{ES}(t) = \frac{A_p}{E_c(t)}\frac{\sigma_c}{1+\frac{E_p}{E_c(t)}(1+\frac{A_p}{A_c}\varepsilon_{cpr}^2)}$ where $\sigma_c = P_{Ag} + (M_{sw} + M_{qp} + P_e)e$</td>
</tr>
<tr>
<td>Combined time-dependent creep, shrinkage and steel relaxation</td>
<td>$\Delta P(t)<em>{C+S+R} = A_p\Delta \sigma</em>{p\text{-c.s.r}}$</td>
</tr>
<tr>
<td></td>
<td>$= A_p\varepsilon_{c+c}\frac{E_p+0.8\Delta \sigma_{p+c}+E_c}{E_c(t)}\varepsilon_{c0}e^{\frac{E_c(t)}{E_c(t)}\varphi(t,t_0)\sigma_{c0}}$</td>
</tr>
<tr>
<td>Autogenous shrinkage of concrete</td>
<td>$\varepsilon_{ca}(t) = \beta_{a,t}(t)\varepsilon_{ca}(\infty)$ where $\varepsilon_{ca}(\infty) = 2.5(f_{ck} - 10) \times 10^{-6}$ and $\beta_{a,t}(t) = 1 - e^{-0.2t^{0.5}}$</td>
</tr>
<tr>
<td>Drying shrinkage of concrete</td>
<td>$\varepsilon_{cd}(t) = \beta_{d,s}(t,t_s)k_h\varepsilon_{cd,0}$ where $\beta_{d,s}(t,t_s) = \frac{t-t_s}{(t-t_s)+0.04\sqrt{h_0}}$ and $\varepsilon_{cd,0} = 0.85 \left(220 + 110\alpha_{ds}\right)e^{-\frac{220}{\sqrt{h_0}}} \times 10^{-6}$ and $k_h$ is a function of $h_0$ (using a lookup table)</td>
</tr>
<tr>
<td>Concrete creep</td>
<td>$\varepsilon_{\infty} = \frac{\sigma_c}{E_{c,eff}}$ where $E_{c,eff} = \frac{E_c}{\varphi(t,t_0)}$ and $\varphi(t,t_0) = \varphi_0\beta_c(t,t_0)$</td>
</tr>
<tr>
<td>Total prestress losses</td>
<td>$\Delta P_{TOT} = \Delta P_{ES} + \Delta P_{C+S+R}$ and $%\text{LOSS} = \Delta P_{TOT}/\Delta P_i$</td>
</tr>
</tbody>
</table>

Continued on next page
Table 5.2 – continued from previous page

* Key notations:
- $A_c =$ cross-sectional area of the concrete section
- $A_p =$ total cross-sectional area of the prestressing strands
- $E_c =$ tangent modulus of elasticity of concrete
- $E_{cem} =$ mean secant modulus of elasticity of concrete
- $e_{cp} =$ eccentricity of the prestressing tendons from the centroid of the section
- $E_p =$ modulus of elasticity of the prestressing strands
- $f_{ck} =$ characteristic cylinder compressive strength of concrete
- $f_{p0} =$ characteristic tensile strength of the prestressing steel
- $h_0 =$ notional size of cross-section ($h_0 = 2A_c/u$)
- $I_c =$ second moment of area of the concrete section
- $t =$ time (for the relaxation loss formula, $t$ is the time after tensioning (hours))
- $u =$ length of perimeter exposed to drying
- $\alpha_{ds1} =$ a coefficient which depends on the type of cement
- $\alpha_{ds2} =$ a coefficient which depends on the type of cement
- $\beta_{as} =$ age of concrete factor for autogenous shrinkage strain
- $\beta_{c}(t,t_0) =$ a coefficient used to describe the development of creep with time after loading
- $\beta_{ds} =$ age of concrete factor for drying shrinkage strain
- $\beta_{RH} =$ relative humidity factor
- $\Delta P_{C+S+R} =$ absolute value of the variation of force in tendons due to creep, shrinkage and relaxation at location x, at time $t$
- $\Delta \sigma_{pr} =$ absolute value of the variation in stress in tendons at location x, at time $t$, due to the relaxation of the prestressing steel
- $\varepsilon_{cs} =$ total shrinkage strain (autogenous + drying) = $\varepsilon_{ca} + \varepsilon_{cd}$
- $\varepsilon_{\infty} =$ final creep strain
- $\sigma_c =$ elastic stress in concrete
- $\sigma_{c,ap} =$ stress in the concrete adjacent to tendons at time $t$, due to self-weight, initial prestress and other quasi-permanent actions
- $\sigma_{pi} =$ stress in the tendons after immediate losses
- $\rho_{1000} =$ relaxation loss 1000 hours after tensioning at 20°C
- $\phi_0 =$ notional creep coefficient
- $\phi(t,t_0) =$ creep coefficient at time $t$ for a load application occurring at time $t_0$

5.2.3 Prestress loss calculation: EC2 time-step method

In order to provide a more refined prediction of prestress loss, an iterative time-step method can be adopted (Hendy and Smith, 2007). The time-step method accounts for the interrelationships between various time-dependent prestress loss mechanisms, the evolution of concrete material properties and the change of total prestressing force over time. A series of iterative calculations were performed between:
5.2 Code predictions of prestress loss

1. Level of total prestress in prestressing tendons at every time step;
2. Relevant input parameters to update at every time step (e.g., material properties, section geometry, loadings);
3. Incremental prestress losses of the time-dependent prestress loss mechanisms at every time step.

Concrete properties, particularly compressive strength and modulus of elasticity, evolve over time as a result of curing and continued hydration. The applied loading and internal stress distribution change over time as a result of staged construction and continued change of the total prestressing force. For example, the casting of concrete deck over the supporting prestressed concrete girders induced stress redistribution as a result of the deck-girder composite action. In this study, a time-step of one day (i.e., 24 hours) was chosen for the EC2 time-step method.

It is expected that the time-step method would give a lower prestress loss prediction compared with the basic method, particularly as a result of lower predictions of creep and relaxation losses. As for relaxation loss prediction in the time-step method, because the level of prestress in prestressing tendons is updated with time to account for continued prestress losses, its prestress loss prediction would be lower than the basic method prediction due to the continuously updated “initial” prestressing force which decreases over time. As for creep loss prediction in the time-step method, because the prestressing force is continuously updated and decreases over time, the associated concrete stress is lower than that from the basic method, and thus the associated creep loss is also lower than that in the basic method prediction. Based on the comparison between the predictions from the EC2 basic method and the EC2 time-step method, the difference between these two predictions is primarily due to the difference in creep loss prediction. Specifically, in terms of percentage prestress loss, the time-step method predictions for TY7 and TYE7 beams are 2.9% and 1.7% lower than the basic method predictions, of which 1.8% and 1.3% are due to the lower creep loss predictions for TY7 and TYE7 beams in the time-step method.

5.2.4 Prestress loss calculation: AASHTO time-step method

The time-step method for prestress loss prediction in AASHTO-LRFD (American Association of State Highway and Transportation Officials [AASHTO], 1998) is tailored for segmental construction (i.e., staged construction) of prestressed concrete girder bridges and takes into account two different stages:

1. From concrete beam casting to deck placement;
2. From deck placement up until the final design life.
Table 5.3 presents a summary of the AASHTO-LRFD time-step method equations for estimating prestress losses. It should be noted that AASHTO-LRFD uses the U.S. customary units with basic units of foot (length), inch (length) and pound (force). Therefore, in this study, the values of input parameters in SI units were first converted to the U.S. customary units to perform the calculations, and the final results (i.e., prestress loss predictions) were then converted back to the SI units for further examination.

Table 5.3 Summary of AASHTO-LRFD formulae for prestress loss predictions using the time-step method.

<table>
<thead>
<tr>
<th>Prestress loss mechanism</th>
<th>AASHTO-LRFD formulae</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Time of transfer to time of deck placement</strong></td>
<td></td>
</tr>
</tbody>
</table>
| Shrinkage of girder concrete | $\Delta f_{pSR} = \varepsilon_{bid}E_p K_{id}$  
where $K_{id} = \frac{1}{1+\frac{E_p A_p}{A_d E_{id}}\left[1+\frac{A_{id}^2}{E_{id}^2}\right][1+0.7\psi_b(t_f,t_i)]}$ |
| Creep of girder concrete | $\Delta f_{pCR} = \frac{E_p}{E_{id}} f_{CRP} \psi_b(t_d,t_i) K_{id}$ |
| Relaxation of prestressing strands | $\Delta f_{pR1} = \frac{f_{pt}}{K_d} \left(\frac{f_{pt}}{f_p} - 0.55\right)$  
and may be assumed to equal to 1.2 ksi for low relaxation strands |
| **Time of deck placement to final time** |
| Shrinkage of girder concrete | $\Delta f_{pSD} = \varepsilon_{ddf} E_p K_{df}$  
where $K_{df} = \frac{1}{1+\frac{E_p A_p}{A_d E_{df}}\left[1+\frac{A_{df}^2}{E_{df}^2}\right][1+0.7\psi_b(t_f,t_d)]}$ |
| Creep of girder concrete | $\Delta f_{pCD} = \frac{E_p}{E_{df}} f_{CRP} \left[\psi_b(t_f,t_d) - \psi_b(t_d,t_i)\right] K_{df} + \frac{E_p}{E_{df}} \Delta f_{cd} \psi_b(t_f,t_d) K_{df}$ |
| Relaxation of prestressing strands | $\Delta f_{pR1} = \Delta f_{pR2}$ |
| Shrinkage of deck concrete | $\Delta f_{pSS} = \frac{E_p}{E_{dc}} \Delta f_{cd} f_{d} K_{df} \left[1+0.7\psi_b(t_f,t_d)\right]$  
where $\Delta f_{cd} = \frac{\varepsilon_{dd} A_d E_{cd}}{1+0.7\psi_b(t_f,t_d)} \left(\frac{1}{A_c} - \frac{e_{pc} e_d}{t_c}\right)$ |
| **Total** |
| Total prestress losses | $\Delta f_{pLT} = (\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR1})_{id} + (\Delta f_{pSD} + \Delta f_{pCD} + \Delta f_{pR2} - \Delta f_{pSS})_{df}$ |

Continued on next page
5.2 Code predictions of prestress loss

Table 5.3 – continued from previous page

* Key notations:
  
  $A_c$ = area of section calculated using the gross composite concrete section properties of the girder and the deck (in.$^2$)
  
  $A_d$ = area of deck concrete (in.$^2$)
  
  $A_g$ = gross area of section (in.$^2$)
  
  $A_{ps}$ = area of prestressing steel (in.$^2$)
  
  $E_{cd}$ = modulus of elasticity of concrete (ksi)
  
  $E_{cd}$ = modulus of elasticity of deck concrete (ksi)
  
  $E_{ci}$ = modulus of elasticity of concrete at transfer (ksi)
  
  $E_p$ = modulus of elasticity of prestressing steel (ksi)
  
  $e_d$ = eccentricity of deck with respect to the gross composite section (in.)
  
  $e_{pc}$ = eccentricity of prestressing force with respect to centroid of composite section (in.)
  
  $e_{pg}$ = eccentricity of prestressing force with respect to centroid of girder (in.)
  
  $I_c$ = moment of inertia of section calculated using the gross composite concrete section properties of the girder and the deck (in.$^4$)
  
  $I_g$ = moment of inertia of the gross concrete section (in.$^4$)
  
  $f_{cg}$ = the concrete stress at the centre of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi)
  
  $f_{pt}$ = stress in prestressing strands immediately after transfer, taken not less than 0.55$f_{py}$
  
  $f_{py}$ = yield strength of prestressing steel (ksi)
  
  $K_L$ = 30 for low relaxation strands and 7 for other prestressing steel, unless more accurate manufacturer’s data are available
  
  $K_{id}$ = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between transfer and deck placement
  
  $K_{df}$ = transformed section coefficient that accounts for time-dependent interaction between concrete and bonded steel in the section being considered for time period between deck placement and final time
  
  $t_d$ = age at deck placement (days)
  
  $t_f$ = final age (days)
  
  $t_i$ = age at transfer (days)
  
  $\Delta f_{cd}$ = change in concrete stress at centroid of prestressing strands due to long-term losses between transfer and deck placement, combined with deck weight and superimposed loads (ksi)
  
  $\Delta f_{pdf} = f_{pdf}$ = change in concrete stress at centroid of prestressing strands due to shrinkage of deck concrete (ksi)
  
  $\varepsilon_{bd}$ = shrinkage strain of girder between time of transfer and deck placement
  
  $\varepsilon_{df}$ = shrinkage strain of deck concrete between placement and final time
  
  $\psi_b(t_d, t_i)$ = girder creep coefficient at time of deck placement due to loading introduced at transfer
  
  $\psi_b(t_f, t_d)$ = girder creep coefficient at final time due to loading at deck placement
  
  $\psi_b(t_f, t_i)$ = girder creep coefficient at final time due to loading introduced at transfer

$(\Delta f_{psR} + \Delta f_{psC} + \Delta f_{psT} )_{id} = \sum$ of time-dependent prestress losses between transfer and deck placement (ksi)

$(\Delta f_{psD} + \Delta f_{psCD} + \Delta f_{psR2} - \Delta f_{psSS} )_{df} = \sum$ of time-dependent prestress losses after deck placement (ksi)
5.2 Code predictions of prestress loss

5.2.5 Assumptions made and sensitivity analysis

A number of assumptions were used in this study when applying the three prestress loss prediction methods described above (sections 5.2.2 to 5.2.4). These assumptions may affect the prestress loss code predictions as well as prestress loss evaluations based on sensor data, and they are summarised as follows:

1. Prestressing tendons and the surrounding concrete were assumed to be perfectly bonded, and therefore any strain changes in prestressing tendons were fully transferred to the surrounding concrete. It was also assumed that the FOS sensors were fully bonded to the surrounding concrete, which would mean that the FOS sensors attached to the prestressing strands were subjected to the same strain changes experienced by the prestressing tendons.

2. Prestress losses due to anchorage set were assumed to be negligible due to the detensioning procedures adopted.

3. The initial prestressing force was assumed to be 209 kN per tendon (corresponding to a stress of 1393 MPa in the wire), based on both the designer’s and the precast facility operator’s calculations.

4. Some material properties of concrete were measured, such as the 7-day and 28-day strengths, as shown in Table 5.1. For estimation of concrete strength evolution in the two time-step methods, the concrete strength value at each date was derived based on EC2 formulae and the measured 28-day strength.

5. It was assumed that concrete cracking was negligible during the early-age of the prestressed concrete beams (e.g., first two years), and thus uncracked gross section properties were used throughout the calculations.

As for input parameter assumptions (e.g., material properties, section geometry, applied loading, environmental conditions), a comprehensive sensitivity analysis was conducted to evaluate the effect of each input parameter on the final prestress loss prediction. Based on the results, the parameters which have relatively significant effects on prestress loss predictions (e.g., a 10% change in the input parameter value results in more than 1% change in percentage prestress loss prediction) include: (i) concrete material properties; (ii) steel material properties; (iii) initial prestressing force; (iv) concrete beam cross-sectional area; and (v) relative humidity. Of these parameters, the values of concrete material properties and relative humidity are particularly uncertain, either because it is not directly measured (in the
case of relative humidity) or because the measurements are highly variable (in the case of concrete material properties).

## 5.3 Prestress loss monitoring results and discussions

### 5.3.1 FOS strain measurements

As mentioned previously in section 4.3 (refer to Figures 4.5 and 4.6 in particular), strain and temperature measurements were taken using Fibre Bragg Grating (FBG) sensors along one top and one bottom prestressing strand for each girder (20 FBG strain sensors and 6 FBG temperature sensors in total for each girder). BOTDR distributed strain measurements were also taken along one top and one bottom prestressing strand for each girder. The strain measurements are plotted in Figures 5.1 to 5.4, which consider a baseline strain equal to those recorded just prior to the transfer of prestress (i.e., detensioning).

![Fig. 5.1 Evolution of the FBG and BOTDR strain profiles at the top and bottom prestressing strands of BM2 (baseline = pre-detensioning, i.e., prior to transfer of prestress).](image)

Fig. 5.1 Evolution of the FBG and BOTDR strain profiles at the top and bottom prestressing strands of BM2 (baseline = pre-detensioning, i.e., prior to transfer of prestress).

Overall, it can be seen from Figures 5.1 to 5.4 that there is a reasonable agreement between the FBG strain data and the BOTDR strain data and the difference between the two sets of monitoring data is mostly less than 100με. However, there are some major discrepancies, which are:

1. The BOTDR data for the top of BM2 and BM3 (i.e., the top prestressing tendon of each TY7 beam) shows much lower strain loss (approximately 400με discrepancy at the
5.3 Prestress loss monitoring results and discussions

Fig. 5.2 Evolution of the FBG and BOTDR strain profiles at the top and bottom prestressing strands of BM3 (baseline = pre-detensioning, i.e., prior to transfer of prestress).

Fig. 5.3 Evolution of the FBG and BOTDR strain profiles at the top and bottom prestressing strands of BM1 (baseline = pre-detensioning, i.e., prior to transfer of prestress).
5.3 Prestress loss monitoring results and discussions

mid-span) and hence much lower prestress loss for the first three months after concrete beam casting compared with the corresponding FBG data (also shown in Butler et al. (2016)). This is clearly anomalous, however, the subsequent measurements of strain loss from the two sets of data (FBG and BOTDR) have close agreement (e.g., both datasets show approximately 400με change at the mid-span from 3 months after casting to pre-service baseline for the top of BM2 and BM3).

2. The BOTDR data of BM1 and BM9 (i.e., TYE7 beams) shows larger strain loss from 9 months onwards after concrete beam casting compared with the corresponding FBG data.

3. For the bottom of BM1, the BOTDR measurements show “strain recovery” from 9 months after concrete beam casting, which is difficult to explain and may be due to erroneous readings.

A discrepancy between distributed and discrete FOS static strain measurements, with the same order of magnitude, was previously reported in a study Sigurdardottir and Glisic (2015). The study used Brillouin Optical Time-Domain Analysis (BOTDA) for distributed strain measurements, which has a higher level of precision (±20με for temperature compensated strain, as specified in Sigurdardottir and Glisic (2015)) compared to BOTDR.

For further data processing and interpretation, the FBG dataset is selected. This is due to a number of observations which add confidence to the accuracy of the FBG dataset:
1. From the FBG dataset, the observed behaviour is consistent within the same type of beam (i.e., comparing BM2 and BM3 and comparing BM1 and BM9) and different between the two types of beams. This is less so for the BOTDR dataset. In particular, it can be seen from the FBG dataset that the prestress loss differences between 3 months after casting and the pre-service baseline are much larger for TY7 internal beams (BM2 and BM3) than for TYE7 edge beams (BM1 and BM9). This makes engineering sense because: (i) the internal beams are under more load than the edge beams due to the loads of ballast, sleepers and rails above the internal beams; and (ii) there is a second stage “beam” casting for the edge beams (200 mm depth × 544 mm width section above each TYE7 edge beam) in the precast facility on Day 55, prior to the in-situ deck casting on Day 185, resulting in lower differential shrinkage and creep.

2. Across all four beams, the top of each beam shows a larger prestress loss from 3 months to 9 months after concrete beam casting compared to the bottom of each beam. This may be due to differential shrinkage as a result of the deck being cast six months after the beams, which would induce more compression at the top of each beam.

3. In addition, this effect of larger prestress loss at the top of each beam is more significant for TY7 beams (BM2 and BM3) than for TYE7 beams (BM1 and BM9). This may be due to the fact that there was already a second stage beam casting for BM1 and BM9 on Day 55 prior to the in-situ deck casting and the strain measurements at 3 months after concrete beam casting, and therefore the effect of differential shrinkage due to deck casting would be smaller for BM1 and BM9.

In summary, although the exact reasons behind the discrepancy between FBG and BOTDR measurements in this study are uncertain, it is likely the result of a combination of effects:

1. The BOTDR data acquisition was affected by frequent errors with the analyser during the transfer of prestress (i.e., detensioning) stage, which was likely due to the cold ambient temperature at the time. In this case, temperature compensation for the BOTDR cables was performed using the FBG temperature measurements. It is also possible that the BOTDR system was giving erroneous strain measurements at the transfer of prestress stage, leading to higher uncertainty in BOTDR strain values. Therefore, the early-age BOTDR data for the top of BM2 and BM3, which is clearly anomalous, may be discarded for further processing and interpretation.

2. Temperature compensation for BOTDR, in particular, may not be accurate due to inaccurate temperature coefficient values, which were assumed to be constant along the BOTDR cables. In addition, the BOTDR measurements have a lower level of precision.
5.3 Prestress loss monitoring results and discussions

(i.e., higher uncertainty level) than the FBG measurements. The higher uncertainty in BOTDR temperature measurements, compared with FBG temperature measurements, results in higher uncertainty in the temperature compensation for the BOTDR cables and thus adds additional uncertainty to the temperature-compensated strain values.

3. Other effects can also contribute the discrepancy, such as the robustness of sensors to their installation environment, the variation in the bonding conditions between the FOS sensors and the concrete surrounding the prestressing strands, the variation in FOS sensor attachment methods, and the variation in strain across the section (note that FBG and BOTDR sensors are attached to different prestressing strands).

Figure 5.5 shows the estimated strain changes at the centroidal level of each girder based on the FBG measurements, with the baseline set at the pre-detensioning stage (i.e., 29 Jan 2015 for TY7 beams and 13 Jan 2015 for TYE7 beams). These are based on the FBG strain readings near the mid-span of each girder. There appears to be some consistent “strain recovery” from the last two sets of data (14 July 2016 and 7 July 2017, corresponding to 4 months in service, and 1 year and 4 months in service, respectively) across BM1, BM2 and BM9. This “strain recovery” can also be observed from the BOTDR strain data (refer to Figures 5.1 to 5.4). This may be due to the effects of seasonal change and in-service loading on the physical response of the bridge such as support settlements. The seasonal influence on strain response as a result of support settlements has been found previously in Abdel-Jaber and Glisic (2019). The sudden drop of prestress level for BM3 at 4 months in service appears to be anomalous, either due to faulty sensor readings or some physical condition change of the girder. Overall, the majority of prestress losses were measured to have occurred during the first three months following the detensioning.

5.3.2 Comparison of code predictions and sensor measurements

Prestress losses were evaluated using the FBG strain data only as the BOTDR strain data has a larger uncertainty level, as discussed previously in section 5.3.1. It should be noted that the values of concrete properties (in particular, concrete strength) used in the evaluation was estimated based on the material test results rather than code values. The measured concrete property values were considered as mean values rather than characteristic values (i.e., 5th percentile). In addition, no partial safety factors for material or loading were applied. This is to enable comparison between unfactored code predictions and sensor measurements. Figure 5.6 presents the results of FBG measured, EC2 predicted and AASHTO predicted prestress losses, with the baseline set at the pre-detensioning stage (i.e., prior to the transfer of prestress).
The prestress loss predictions using the two time-step methods are lower than those using the basic method. It appears from Figure 5.6 that prestress loss predictions using the EC2 basic method, compared with those using the EC2 time-step method, have closer agreement with the measured prestress losses in terms of magnitude. However, based on the sensitivity analysis, inaccurate input parameter values can have a noticeable influence on code predictions. While prestress loss predictions from the EC2 basic method continue to grow at a relatively fast rate (based on the predicted final prestress loss prediction (%), i.e., the total prestress loss at the design life), both the measured prestress losses and the prestress loss predictions using the time-step methods begin to level off at an earlier point in time. The reason for the anomalous readings after 500 days for BM3 (particularly the sudden increase in prestress loss) is not known, and may be due to data error or actual physical changes to the beam. The reason for the “strain recovery” from the last two sets of data (at 539 days and 897 days after casting for TY7 beams, and at 552 days and 910 days after casting for TYE7 beams) has previously been discussed in the last paragraph of section 5.3.1. Overall, the unfactored code predictions of prestress loss are slightly lower than the prestress losses back-calculated from the sensor data, but the discrepancy is generally less than 1% prestress loss.
5.3 Prestress loss monitoring results and discussions

Fig. 5.6 Evolution of prestress losses with time for the instrumented TY7 and TYE7 beams: (a) TY7 beams; and (b) TYE7 beams.
5.3.3 Effect of differential shrinkage

Differential shrinkage resulting from the staged construction can cause additional prestress loss. Due to the difference in age between the deck slab concrete and the girder concrete, the girder would already have experienced a significant amount of concrete shrinkage prior to the deck casting and therefore would undergo shrinkage at a lower rate than the deck concrete. As the in-situ concrete deck would experience larger shrinkage strain compared to the precast concrete girder (i.e., creating differential shrinkage strains), a compressive force would be exerted along the top surface of the concrete girder, resulting in additional prestress loss in the tendons.

Based on Hendy and Smith (2007), the effect of differential shrinkage can be calculated using Equations 5.2 and 5.3 below:

\[
\varepsilon_{diff} = \varepsilon_{sh,\text{slab}}(\infty) - \left( \varepsilon_{sh,\text{beam}}(\infty) - \varepsilon_{sh,\text{beam}}(t_1) \right)
\]

\[
F_{sh} = \varepsilon_{diff} EA_{\text{slab}} \left(1 - e^{-\phi} \right) / \phi
\]

where

- \( \varepsilon_{diff} \) = differential shrinkage strain
- \( \varepsilon_{sh,\text{slab}}(\infty) \) = total shrinkage strain of the slab
- \( \varepsilon_{sh,\text{beam}}(\infty) \) = total shrinkage strain of the precast beam
- \( \varepsilon_{sh,\text{beam}}(t_1) \) = shrinkage strain of the precast beam after casting the slab
- \( F_{sh} \) = axial restrained force
- \( E \) = modulus of elasticity of concrete deck slab
- \( A_{\text{slab}} \) = cross-sectional area of concrete deck slab
- \( \phi \) = creep ratio

The predicted effect of differential shrinkage for the beams in this study is shown in Table 5.4. Note that unlike TY7 internal beams, the TYE7 edge beams had a second stage beam casting on Day 55 after the initial concrete beam casting. It can be seen from Table 5.4 that the effect of differential shrinkage on prestress loss predictions for the TY7 and TYE7 beams were found to be negligible in this case study.
Table 5.4 Effect of differential shrinkage.

<table>
<thead>
<tr>
<th>Additional prestress losses due to differential shrinkage</th>
<th>EC2 method based on Hendy and Smith (2007)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TY7 beams</td>
</tr>
<tr>
<td>1 year and 4 months in-service</td>
<td>2.9 MPa</td>
</tr>
<tr>
<td>(Day 897 for TY7 beams and Day 910 for TYE7 beams, after beam casting)</td>
<td>(0.21%)</td>
</tr>
<tr>
<td>120 years</td>
<td>5.4 MPa</td>
</tr>
<tr>
<td>(Day 43,800 after beam casting)</td>
<td>(0.39%)</td>
</tr>
</tbody>
</table>

5.4 Discussion

Compared with the existing body of literature on prestress loss monitoring, this study serves as a new case study which:

1. Provides both discrete (FBG) and distributed (BOTDR) fibre optic sensor measurements of relatively long-term behaviour of prestress loss (two and a half years since concrete beam casting), and compares and assesses the accuracy of these two sets of data

2. Compares measured prestress losses with predictions from code-based methods, which is informed by sensitivity analysis, and provides engineering interpretation of the measured prestress loss behaviour

The results presented in this study have demonstrated the feasibility of monitoring and evaluating prestress loss, a critical parameter in the structural modelling of prestressed concrete bridges, to enable more realistic capacity assessment of this type of bridge in practice. The methodology in this study can be applied to other newly-constructed prestressed concrete bridges.

5.5 Summary

This chapter has evaluated the prestress loss behaviour (over two and a half years) of four prestressed concrete beams of the Chebsey bridge. Both discrete (FBG) and distributed (BOTDR) fibre optic sensor datasets were presented. The unfactored code predictions of prestress losses were compared with the sensor measurements based on the FBG dataset.
Three code-based methods were used to predict long-term prestress losses: one basic method (based on EC2) and two time-step methods (based on EC2 and AASHTO-LRFD). A summary of the key findings is listed below:

- The prestress loss predictions using the two time-step methods (one based on EC2 with a time-step of one day and one based on AASHTO-LRFD with time steps of different construction stages) are lower than those using the basic method (based on EC2), which is as expected.

- There is a reasonable agreement between the FBG and the BOTDR datasets (the difference is mostly less than 100 $\mu\varepsilon$), although there are some noticeable discrepancies. The discrepancies may be due to a combination of factors: (i) the frequent errors with the analyser during the BOTDR data acquisition at the transfer of prestress stage; (ii) inaccurate temperature compensation for BOTDR due to inaccurate temperature coefficient values; (iii) sensor robustness; (iv) variation in the FOS sensor attachment methods; and (v) variation in strain across the section (FBG and BOTDR sensors are attached to different prestressing strands).

- Based on sensitivity analysis, it has been found that concrete material properties, steel material properties, initial prestressing force, concrete beam cross-sectional area and relative humidity can have relatively significant influence on the final prestress loss predictions (e.g., a 10% change in the input parameter value results in more than 1% change in percentage prestress loss prediction).

- It has been found that unfactored code predictions slightly underestimate the FOS measurements of prestress loss (generally by less than 1% prestress loss) but overall, there is a close agreement, particularly for medium- and long-term prestress losses (two years after concrete beam casting). In addition, based on the findings of this study, both the measured prestress losses and the prestress loss predictions using the time-step methods (based on EC2 and AASHTO-LRFD) begin to level off at an earlier point in time compared with the prestress loss predictions from the EC2 basic method.

- Both the prediction models and the sensor measurements indicate that the majority of the prestress losses occurred during the first six months following the concrete beam casting.

- In this case study, it has been found that the effect of differential shrinkage between the concrete girders and the cast-in-situ concrete deck slab on the change in prestress loss was negligible for the Chebsey bridge (less than 0.5% prestress loss).
5.5 Summary

- Overall, it has been demonstrated that prestress loss, a critical parameter in structural modelling and capacity assessment of prestressed concrete bridges, can be monitored and evaluated. Specifically, the information of prestress loss can be used to more realistically evaluate the level of prestress in prestressed concrete bridges and thus enable more realistic assessment of safety (e.g., load capacity) and serviceability (e.g., concrete cracking, in-service deflections).
Chapter 6

Structural Behaviour Part II – Behaviour under Live Load

6.1 Introduction

This chapter presents the evaluation of another important aspect of the structural behaviour of the Chebsey bridge: behaviour under live load. In particular, it investigates how monitoring data may be processed to better understand and quantify commonly made assumptions about in-service structural behaviour of bridges. This is to enable more realistic structural modelling and analysis for bridge assessment purposes.

Specifically, this study investigates three commonly adopted modelling assumptions about bridge behaviour under live load using the monitoring data collected:

1. **Contribution of secondary elements**
   Secondary elements (e.g., bridge parapets, surfacing layer) are commonly assumed to have no stiffness or strength contribution in bridge design and assessment.

2. **Load distribution characteristics**
   These are commonly assumed using the values of load distribution factor (DF) recommended in codes and standards.

3. **Support boundary conditions**
   These are commonly assumed as being simply supported for bridges with simple supports, which potentially result in conservative (high) predictions of bending moments along the beams.
Both direct interpretation of the monitoring datasets themselves (section 6.2) and comparison of monitoring data with model predictions (in particular, structural model updating) (section 6.3) were used to enable better engineering interpretation and understanding.

It is envisaged that the methodology and workflow in this study will be portable to a portfolio of multi-girder concrete bridges (reinforced or prestressed), enabling more realistic structural modelling and assessment and thus facilitating more targeted asset management of these bridges.

### 6.2 Direct interpretation of the monitoring datasets

#### 6.2.1 Data pre-processing

In this study, the FBG measured strain response under six train-passage events on 27th September 2017 was used (refer to Table 4.1 for a summary of data collection). This date corresponds to a period of approximately one year and four months of in-service life for the bridge and approximately two and a half years since concrete beam casting. The six trains include four multi-car freight trains (denoted as Trains No. 1, 2, 5 and 6) and two 4-car passenger trains (denoted as Trains No. 3 and 4). Train No. 3 is a British Rail Class 350 northbound passenger train. This train’s nominal axle loads (unladen) were given by Network Rail (shown in Figure 6.8 in section 6.3.1 later). There was no information available on the axle loads of the other five trains.

Changes of wavelength measured using the FBG sensors were first converted to changes in strain using Equation 4.2 in Chapter 4, based on the assumption that changes in ambient temperature during each train-passage event were negligible. The adopted sampling rate for the FBG system was 125 Hz. Before data post-processing and interpretation, the noise level of the dynamic strain data was estimated by calculating the standard deviation of the data under ambient conditions (i.e., no train load and little change of environmental condition). It was found that the background noise level of the strain data was approximately ±2µε (corresponding to ±2 standard deviations) across all FBG sensors.

Given that the measured peak strain response during a train-passage event was typically below 20µε magnitude in this study, the data was denoised to increase the signal-to-noise ratio (SNR) and thus minimise the effects of data noise and outlier measurement (if there is any) on data post-processing and interpretation. An N = 5 zero-centred moving average filter (where N is the number of data points for which the average is calculated) was applied. As mentioned previously, the data frequency adopted was 125 Hz, which means that the moving average filter calculated the average over every 1/25th of a second (corresponding
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To every 5 data points. The data noise level after applying the moving average filter was approximately $\pm 0.8 \mu \varepsilon$ (corresponding to $\pm 2$ standard deviations) across all FBG sensors. Figure 6.1 depicts the typical raw and filtered time history strain response of the bridge recorded during train-passage events. Table 6.1 summarises the peak strain responses under the train-passage events investigated in this study.

Table 6.1 Train type and peak strain response (after applying the moving average filter; +ve is tensile and –ve is compressive).

<table>
<thead>
<tr>
<th>Train number</th>
<th>Train type</th>
<th>Peak strain response at BM5 L5 (near mid-span)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Top sensor</td>
</tr>
<tr>
<td>1</td>
<td>Freight</td>
<td>$-5.0\mu \varepsilon$</td>
</tr>
<tr>
<td>2</td>
<td>Freight</td>
<td>$-6.8\mu \varepsilon$</td>
</tr>
<tr>
<td>3</td>
<td>Passenger</td>
<td>$-3.3\mu \varepsilon$</td>
</tr>
<tr>
<td>4</td>
<td>Passenger</td>
<td>$-3.0\mu \varepsilon$</td>
</tr>
<tr>
<td>5</td>
<td>Freight</td>
<td>$-6.7\mu \varepsilon$</td>
</tr>
<tr>
<td>6</td>
<td>Freight</td>
<td>$-7.3\mu \varepsilon$</td>
</tr>
</tbody>
</table>

6.2.2 Prestressed concrete girder neutral axis: inferring contribution of secondary elements

As shown previously in Figure 4.5a, one top and one bottom prestressing strands of each longitudinal girder were instrumented. The measured live load responses under the six train-passage events were found to be small (refer to Table 6.1) and well within the serviceability limit state (SLS). This means that the bridge was under linear elastic behaviour. Therefore, the neutral axis at each measured cross-section, $z_{N.A.}$, could be estimated based on the corresponding top and bottom sensor measurements by assuming a linear strain profile and performing linear interpolation, using Equation 6.1:

$$\frac{z_{N.A.} - z_{bottom}}{z_{top} - z_{bottom}} = \frac{0 - \Delta \varepsilon_{bottom}}{\Delta \varepsilon_{top} - \Delta \varepsilon_{bottom}}$$

(6.1)

where $z$ is the vertical distance above the beam soffit and $\Delta \varepsilon$ is the measured change of strain during a train-passage event. To obtain more precise estimates of neutral axis, strain data under the passage of Train No. 6 (a multi-car freight train) was used as it gave the
Fig. 6.1 Raw and filtered (N = 5 zero-centred moving average) time history response at BM5 L5 bottom sensor location: (a) under Train No. 3 (British Rail Class 350) passage event; and (b) under Train No. 6 (the heaviest freight train measured) passage event.
largest magnitude of strain response and thus the largest signal-to-noise ratio (SNR), thereby minimising the uncertainty level of the neutral axis estimates.

Figure 6.2 provides a summary of the neutral axis results. Figure 6.2a shows the variation in computed neutral axis with time for BM5 at L5 (near mid-span) before, during and after the passage of Train No. 6. The train-passage event lasted approximately 15 seconds. It can be seen that the uncertainty level of the neutral axis estimates varies with time. In particular, there are very large variations before and after the train-passage event and a series of 20 large local variations (local peaks and troughs) during the 15 seconds train-passage time window. These 20 large local variations during the 15 seconds train-passage event correspond to the short time windows of small loading occurring between the passage of one train axle and the next. Because of the low strain magnitude under such loading (refer to Figure 6.1b for a plot of strain time history), the SNR is low and thus, the uncertainty level of the neutral axis estimate is high, resulting in these large local variations. This was explained in more detail in an uncertainty analysis Sigurdardottir and Glisic (2013). Similarly, this also explains the very large variations before and after the train-passage event when the dynamic strains are extremely small.

To minimise the effect of these large local variations (i.e., high uncertainty levels) on the final estimation of neutral axis for each location, a threshold check was performed on the magnitude of strain data to remove neutral axis estimations with high uncertainty levels due to low strain magnitude and thus low SNR. As mentioned previously, the strain data noise after applying the moving average filter is around $\pm 0.8\mu\varepsilon$. Based on this value and trial-and-error, in this study, neutral axis estimations from strain data with $|\Delta\varepsilon_{top} - \Delta\varepsilon_{bottom}| < 15\mu\varepsilon$ were removed, as illustrated in Figure 6.2a. Using this method, the neutral axis positions at several longitudinal locations corresponding to the positions of the FBG sensors were obtained for the six instrumented girders, as shown in Figure 6.2b (for L5 only, i.e., near mid-span) and Figure 6.2c (for L3 to L8). Table 6.2 provides a summary of the results in Figure 6.2c.

Table 6.2 Neutral axis results for the six instrumented girders (based on the mean value of L3 to L8 results for each girder).

<table>
<thead>
<tr>
<th>Beam</th>
<th>BM9</th>
<th>BM5</th>
<th>BM4</th>
<th>BM3</th>
<th>BM2</th>
<th>BM1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neutral axis (mm)</td>
<td>484</td>
<td>467</td>
<td>471</td>
<td>473</td>
<td>474</td>
<td>508</td>
</tr>
</tbody>
</table>

It can be seen from Figures 6.2b and 6.2c and Table 6.2 that the neutral axis positions for the edge TYE7 beams (especially BM1) are distinctively higher than that for the internal TY7 beams. This is found to be consistent at different longitudinal locations (L3 to L8).
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Fig. 6.2 Neutral axis results for the six instrumented girders: (a) time series plot of neutral axis results of BM5 mid-span under Train No. 6 passage event; (b) neutral axis at L5: showing mean and variation (± 1 standard deviation) – left to right: BM9, BM5 to BM1; and (c) neutral axis levels at L3 to L8: showing mean values – left to right: BM9, BM5 to BM1.
Since the bridge was newly constructed and no concrete cracking was observed at the deck soffit at the time of data collection (1 year and 4 months in service), this difference in neutral axis is likely the result of the secondary elements (in particular, bridge parapets above the edge beams) being partially bonded to the bridge deck (refer to Figure 4.2a in Chapter 4), forming a partial composite action. A similar effect of partial composite action on neutral axis position has previously been found and investigated under other loading scenarios (e.g., controlled load tests, thermal load) (Algohi et al., 2020; Yarnold et al., 2018). The effect of this partial composite action is more significant for the edge girders (BM1 and BM9) as the parapets are directly above these two girders and they are bonded through the dowel bars right next to these two girders (refer to Figure 4.2a in Chapter 4).

In this case, the total depth of the bridge deck (cast in-situ deck slab and precast girder) is 900 mm. Since the difference in modulus of elasticity between the precast and the cast in-situ components was found to be negligible, it is expected that the neutral axis position is approximately 450 mm from the deck soffit. An extreme scenario of the bridge parapets being fully bonded to the solid slab bridge deck would give an average neutral axis position of 645 mm across the nine girders. Another extreme scenario of each bridge parapet being fully bonded only to the corresponding edge girder (i.e., considering a composite section of edge girder and bridge parapet) would give an edge girder neutral axis position of 940 mm. Therefore, based on these two extreme scenarios and assuming no concrete cracking, it can be inferred that the degree of partial composite action between the parapets and the bridge deck is very low, and therefore the stiffness contribution from the bridge parapets is very small.

In addition, based on the results in Figure 6.2c and Table 6.2, the estimated neutral axis positions of the internal TY7 beams are around 470 mm, which is 20 mm higher than the theoretical value of 450 mm. Moreover, the neutral axis level is lowest at BM5 and slightly higher closer to the edge beam BM1. Given that the bridge was in good condition (newly constructed and no concrete cracking observed) at the time of data collection and the deck surfacing layer consists of only 25 mm waterproofing layer (which has a much lower modulus of elasticity compared to concrete), this 20 mm discrepancy is likely the result of the bridge parapets being partially bonded to the bridge deck. However, this stiffness contribution from the secondary elements to the internal TY7 girders is extremely small and therefore may be ignored.

Overall, it has been demonstrated from this study that neutral axis results under live load may be used to infer the contribution of secondary elements (e.g., bridge parapets, surfacing layer) to structural stiffness. This can then be used to enable more realistic serviceability limit state (SLS) assessment (e.g., stress, deflection). It should be noted that the contribution of
6.2 Direct interpretation of the monitoring datasets

secondary elements to structural strength is not guaranteed and require further investigation of the individual bridge of interest, as the degree of partial composite action may start to break down under cyclic loads over time or as the load increases towards the ultimate limit state (ULS) (Bakht and Jaeger, 1992).

6.2.3 Longitudinal bending: normalised curvature profile (relative to central beam) and moment envelope

The real-time longitudinal bending curvatures at each longitudinal location along each prestressed concrete girder were computed based on the corresponding top and bottom FBG sensor measurements, using Equation 6.2:

\[ \Delta \kappa = \frac{\Delta \varepsilon_{\text{top}} - \Delta \varepsilon_{\text{bottom}}}{z_{\text{top}} - z_{\text{bottom}}} \]  

(6.2)

where \( \Delta \kappa \) is the change of curvature, \( z \) is the vertical distance above the beam soffit and \( \Delta \varepsilon \) is the measured change of strain during a train-passage event. It should be noted that only one sensor at L10 position was working for each instrumented beam (specifically, top sensor at \( z = 600 \) mm for BM1 and BM3 and bottom sensor at \( z = 100 \) mm for BM2, BM4, BM5 and BM9) and thus the curvature at L10 cannot be calculated using Equation 6.2.

To enable comparison of structural response across different girders during a train-passage event, the curvature profiles of the six instrumented girders (BM1 to BM5, BM9) were normalised relative to the curvature profile of the central girder BM5, using Equation 6.3:

\[ \kappa_{\text{normalised relative to BM5,BMi}}(x) = \frac{\kappa_{\text{BMi}}(x)}{\kappa_{\text{BM5}}(x)} \]  

(6.3)

where \( x \) is the longitudinal location along each beam and \( i \) is the beam number (\( i = 1, 2, 3, 4, 5, 9 \)). Normalised curvature was computed to help characterise and evaluate the bridge deck behaviour under train loads, and in particular, the load distribution across different girders.

Figure 6.3 provides a summary of the normalised curvature results for the instrumented girders. Figure 6.3a shows a time series plot of the computed normalised curvature for BM4 at L5 (relative to BM5 at L5, i.e., near mid-span) before, during and after the passage of Train No. 6. Similar to the post-processing of neutral axis results in Figure 6.2 (refer to section 6.2.2), to minimise the effects of the large local variations (i.e., high uncertainty levels) on the normalised curvature estimate due to low strain magnitude and low SNR, a threshold check was performed on the magnitude of strain data to remove normalised curvature estimations with high uncertainty levels. Specifically, normalised curvature estimations
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from strain data with $|\Delta e_{\text{top}} - \Delta e_{\text{bottom}}| < 15 \mu e$ for BM5 were removed, as illustrated in Figure 6.3a. The normalised curvature results for all six instrumented girders (at L3 to L8) are shown in Figure 6.3b (for Train No. 6 passage event) and Figure 6.3c (for Train No. 5 passage event).

It can be seen from Figures 6.3b and 6.3c that the longitudinal bending curvature profile appears to be near uniform across the six instrumented girders. It can also be seen that Figures 6.3b and 6.3c are extremely similar, suggesting that the normalised curvature profile depends little on the train loads. This is as expected since the normalised curvature represents the underlying load sharing behaviour which is an intrinsic property of the bridge deck.

Based on the curvature results, longitudinal bending moment ($M$) may be estimated, using Equation 6.4:

$$M = EI\Delta \kappa$$

where $E$ may be taken as the tangent modulus of concrete at the time of the train-passage event, and $I$ is the second moment of area of the cross-section of interest. For the Chebsey bridge, strength class C60/75 concrete was used for all prestressed concrete beams during construction (Butler et al., 2016). Based on Eurocode 2, the tangent modulus of C60/75 concrete at the time of data collection was estimated to be 44 MPa (approximately one year and four months in service, approximately two and a half years since concrete casting). In addition, the difference in modulus of elasticity between precast concrete beams and in-situ concrete infill deck slab is very small and thus assumed to be negligible.

The second moment of area was calculated for a longitudinal grillage beam section of the composite solid slab deck. The grillage beam section was defined based on Hambly (1991). Details of the grillage modelling of the Chebsey bridge are provided in the next section (section 6.3). Each longitudinal grillage beam is comprised of one prestressed concrete girder as well as the surrounding cast in-situ concrete infill deck slab. It has a cross-section of 750 mm width (beam spacing) and 900 mm depth (700 mm beam depth plus 200 mm slab depth). In addition, the section was assumed to be uncracked at typical service loads and therefore, gross section properties were used to calculate the second moment of area.

Using the method described above, the bending moment and the moment envelope of each composite solid longitudinal grillage beam (750 mm × 900 mm cross-section) were calculated for each train-passage event. Figure 6.4 shows the moment envelope results for each prestressed concrete beam (composite solid section) under Train No. 3 (Class 350 4-car passenger train) passage event based on the FBG measurements. The noise level for the moment estimations was found to be approximately ±7 kNm (±2 standard deviations), based on the moment results under ambient conditions (i.e., no train-passage event). Based on
Fig. 6.3 Normalised curvature profile (normalised relative to BM5): (a) time series plot of normalised curvature results of BM4 mid-span (relative to BM5 mid-span) under Train No. 6 passage event; (b) under Train No. 6 passage event (mean and ±1 standard deviation) – left to right: BM9, BM5 to BM1; top to bottom: L8 to L3; and (c) under Train No. 5 passage event (mean and ±1 standard deviation) – left to right: BM9, BM5 to BM1; top to bottom: L8 to L3.
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the results presented in Figures 6.4c and 6.4d, no noticeable hogging moment was detected at the sensor locations (which are some distance away from the end supports, as shown in Figure 4.5a in Chapter 4) as the hogging moment envelope was within the moment noise level. For the sagging moment envelope, as shown in Figures 6.4a and 6.4b, no significant variation across different beams was observed, suggesting near uniform transverse load distribution across the beams. This suggests that the bridge deck is very stiff transversely and acts predominantly as a solid beam (i.e., one-way bending).

![Fig. 6.4 Longitudinal bending moment envelope for the prestressed beams (composite section) during Train No. 3 passage event (sagging moment taken as negative): (a) sagging moment envelope: longitudinal profile; (b) sagging moment envelope: transverse profile; (c) hogging moment envelope: longitudinal profile; and (d) hogging moment envelope: transverse profile.](image)

6.2.4 Transverse bending: strain at top reinforcement and transverse steel cross tie

As previously shown in Figure 4.5b in Chapter 4, one top transverse reinforcing bar (at \( z = 840 \text{ mm} \)) of the RC deck slab and one transverse steel cross tie (at \( z = 280 \text{ mm} \)) were instrumented at the mid-span and the east quarter span locations. These can be used to
evaluate the transverse bending behaviour of the bridge deck. Figure 6.5 shows the transverse strain profile at the instances of the largest magnitude of tensile and compressive responses.

![Fig. 6.5 Transverse strain profile at the instances of highest magnitude of tensile and compressive strains: (a) deck slab top reinforcement at mid-span; and (b) transverse steel cross tie at mid-span.](image)

The transverse bending behaviour is shown clearly in Figure 6.5. Based on the strain data, the top reinforcement is in compression near the centre and in tension near the two sides during a train-passage event, although the magnitude of strain is very small. The maximum tensile strain of the top reinforcement bar near BM5 (i.e., the central longitudinal beam) was approximately $5 \mu \varepsilon$ (tensile strain) and approximately $2.5 \mu \varepsilon$ (tensile strain) during the passage of Train No. 6 and Train No. 3, respectively. The measured strain profile for the steel cross tie was negligible (generally less than $2 \mu \varepsilon$) under both train-passage events. Overall, it can be seen that although there was some transverse bending, the magnitude was extremely small. This suggests that the bridge deck has a very high transverse bending stiffness and thus acts predominantly as a rigid beam. This results in close to uniform load sharing across different girders, which agrees well with the findings from the longitudinal bending moment results as shown previously in section 6.2.3.

### 6.2.5 Load distribution factor

To evaluate how the train load is shared across different girders, a load distribution factor (DF) may be calculated. In this study, the load distribution factor is defined as the ratio between the moment at a girder BM$i$ (where $i$ is the beam number) and the sum of moments
6.2 Direct interpretation of the monitoring datasets

across all girders, as shown in Equation 6.5:

\[ DF_{BMi} = \frac{M_{BMi}}{\sum M_{BMi}} \]  

(6.5)

Using this formula, a load distribution factor of 1/9 across all nine girders indicates uniform load sharing across the nine longitudinal girders. In this study, since only six of the nine girders were instrumented, \( \sum M_{BMi} \) was calculated based on the measurements of BM1 to BM5 (i.e., half of the deck) and the assumption of symmetric behaviour. BM9 measurements were used to check symmetry in deck behaviour.

Figure 6.6 provides a summary of the load distribution factor results, which are compared against the uniform load distribution scenario and the design prediction of transverse load distribution factor based on Morice and Little (1955). It can be seen that overall, the load distribution factors are close to 1/9, suggesting near uniform transverse load distribution across the girders. The measured DF is distinctively smaller than the typical design DF based on Morice and Little (1955) (smaller by approximately 10%). Additionally, the edge TYE7 beam BM1 appears to have slightly lower moment results, which may be due to the fact that BM1 is partially bonded to the parapet above (refer to section 6.2.2 and Figure 6.2) and thus the partially bonded parapet carries some moment, resulting in lower moment carried by the beam BM1 itself.

![Fig. 6.6 Load distribution factor across different girders (based on the average of L5 and L6 moments, i.e., close to mid-span) – left to right: BM9, BM5 to BM1.](image-url)
6.3 Comparison of monitoring data with model predictions

6.3.1 Benchmark model

To model and analyse the structural behaviour of the bridge deck in both longitudinal and transverse directions, a grillage analysis was performed. The grillage analysis method is one of the most commonly used methods for modelling and analysing deck behaviour in bridge design and assessment, particularly for multi-girder bridges. For example, in the U.K., bridge assessment in industry practice typically starts from a low fidelity model (e.g., line beam model) and proceeds to higher fidelity models (e.g., grillage model, finite element model) until the assessed structural capacity is satisfactory or else interventions are deemed necessary (Highways England, 2019). Compared with grillage models, finite element (FE) models are less frequently used for bridge operation and maintenance (O&M) in practice. The following analyses (sections 6.3.2 to 6.3.4) demonstrate how strain monitoring data may be used to enable more realistic grillage modelling. An FE model was also created to compare with the grillage model results (refer to section 6.3.2 and Table 6.6 for more details).

A benchmark grillage model was first created using Oasys GSA software (version 8.7). In the longitudinal direction, the deck was divided into nine longitudinal grillage beams, each with a rectangular cross-section of 750 mm × 900 mm (where 750 mm is the beam spacing and 900 mm is the total depth of slab (200 mm) and girder (700 mm)). Each longitudinal grillage beam represents one longitudinal girder as well as the surrounding cast in-situ concrete infill deck slab. In the transverse direction, the deck was divided into 17 transverse grillage beams, each with a cross-section of 700 mm × 765 mm (where 700 mm is 1/17th of the girder length (11.9 m) and 1/16th of the bridge span (11.2 m); and 765 mm is the total depth of the in-situ concrete infill). The benchmark model ignores any contribution of secondary elements (e.g., parapets) to structural stiffness and strength, a typical assumption made in bridge design and assessment. Material properties of C60/75 concrete were used based on the construction information. Based on EC2 code values of concrete material properties, the effect of the differing moduli between precast concrete girders and in-situ concrete infill deck slab on stiffness properties was found to be negligible.

The benchmark model used simply supported boundary conditions. This was assumed in the original design due to the use of elastomeric bearings for the two end supports. However, due to the use of solid concrete infill diaphragms at both ends of the girders (refer to Figure 4.2b in Chapter 4), the actual boundary conditions are expected to be partially fixed with some degree of rotational restraint. This was modelled by introducing rotational springs (rotating about the transverse axis) at the supports. The rotational spring stiffness, $K_r$, was...
assigned as 0 for each support in the benchmark model (i.e., simply supported). Figure 6.7 shows the completed grillage model of the bridge deck.

![Elastomeric bearings at supports](image)

Fig. 6.7 Grillage model for the bridge deck of the Chebsey bridge in Oasys GSA.

As for the effects of train loads, two load cases were analysed. One was the Eurocode 1 Load Model 71 (LM71), which was used as the design load case for standard railway loads. The other was the nominal axle loads (unladen) of British Rail Class 350 (C350), as shown in Figure 6.8, which is the train type of Train No. 3 (a passenger train) in this study. This was used to enable comparison of model predicted and sensor measured live load effects in the subsequent model validation and updating process.

Based on Eurocode 1 (BS EN 1991–2:2003, Clause 6.4.4) (CEN/TC250, 2003), a dynamic analysis is not needed for this bridge. Previous monitoring studies on similar types of bridge under similar types of train load found that the effect of variations in train speed and passenger load on the structural response was small (Acikgoz et al., 2018; Lin et al., 2019). Therefore, in the analysis, the C350 nominal axle loads were applied statically (i.e., without any dynamic amplification factor) as successive moving loads at different longitudinal locations along the bridge to simulate the structural responses under the Train No. 3 passage event.

To compare the monitoring data with model predictions of live load response, Table 6.3 shows the moment envelope derived from the FBG sensor measurements and Table 6.4 shows the moment envelope predicted from the benchmark model, both under the passage of Train No. 3 (C350). Table 6.5 includes both sets of data in one table for direct comparison. It can be seen from these two sets of data that the moment envelope results from the benchmark model are much higher than those based on FBG sensor measurements (e.g., maximum
6.3 Comparison of monitoring data with model predictions

moment from the model is approximately 50% higher than that based on the sensor data). This suggests that certain model assumptions or parameters need updating.

6.3.2 Sensitivity analysis

In general, any discrepancies between model predictions and sensor measurements could be due to model parameter uncertainties (e.g., material properties, geometric properties, boundary conditions, loading), modelling uncertainties (i.e., model structure uncertainties: e.g., model type, mesh size) and data uncertainties (e.g., data precision, data accuracy). To systematically investigate which factors may contribute to the model-data discrepancy, a comprehensive sensitivity analysis of model parameters and modelling assumptions (or modelling methods) was performed, as summarised in Table 6.6. In addition, the bending moment derived from FBG sensor measurements was computed using \( M = EI\Delta \kappa \) (refer to section 6.2.3 and Equation 6.4), which is proportional to the assumed \( E \) and \( I \) values. Based on this and the sensitivity analysis results summarised in Table 6.6, the main potential sources of uncertainty behind the model-data discrepancy in this study have been identified. These include:

1. Support boundary conditions (e.g., rotational spring stiffness, \( K_r \))
2. Input axle loads
3. Concrete modulus of elasticity (\( E_c \)) (affecting the moment envelope derived from FBG sensor measurements as \( E \) needs to be assumed when calculating moment)
4. Stiffness contribution of secondary elements such as parapets (affecting the second moment of area, \( I \))

These are further examined and discussed in the following subsections (6.3.3 and 6.3.4) on model updating and model-data comparison, particularly support boundary conditions and the contribution of secondary elements.
6.3 Comparison of monitoring data with model predictions

Table 6.3 Estimated sagging moment envelope under Train No. 3 (British Rail Class 350) passage event based on FBG measurement data.

<table>
<thead>
<tr>
<th></th>
<th>BM1</th>
<th>BM2</th>
<th>BM3</th>
<th>BM4</th>
<th>BM5</th>
<th>BM9</th>
<th>Mean</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L1</td>
<td>L2</td>
<td>L3</td>
<td>L4</td>
<td>L5</td>
<td>L6</td>
<td>L7</td>
</tr>
<tr>
<td>[kNm]</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>-19.5</td>
<td>-23.2</td>
<td>-33.2</td>
<td>-39.5</td>
<td>-38.3</td>
<td>-39.5</td>
<td>-36.0</td>
</tr>
<tr>
<td>BM1</td>
<td>-17.5</td>
<td>-26.1</td>
<td>-33.8</td>
<td>-44.6</td>
<td>-40.7</td>
<td>-45.5</td>
<td>-45.5</td>
</tr>
<tr>
<td>BM2</td>
<td>-12.5</td>
<td>-22.2</td>
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<td>-40.8</td>
<td>-45.5</td>
<td>-43.9</td>
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<td>-45.8</td>
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<td>-25.2</td>
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<td>-39.7</td>
<td>-39.4</td>
<td>-44.9</td>
<td>-41.8</td>
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<tr>
<td>BM9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mean</td>
<td>-14.1</td>
<td>-23.5</td>
<td>-33.1</td>
<td>-40.3</td>
<td>-41.7</td>
<td>-43.5</td>
<td>-42.6</td>
</tr>
</tbody>
</table>

* Note: Data from only one sensor was available for each beam at L10 (top sensor at z = 600 mm for BM1 and BM3 and bottom sensor at z = 100 mm for BM2, BM4, BM5 and BM9) and thus curvature and moment results at L10 are not available.

Table 6.4 Sagging moment envelope results under Train No. 3 (British Rail Class 350) passage event from benchmark grillage model.

<table>
<thead>
<tr>
<th></th>
<th>BM1</th>
<th>BM2</th>
<th>BM3</th>
<th>BM4</th>
<th>BM5</th>
<th>BM9</th>
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<tbody>
<tr>
<td></td>
<td>L1</td>
<td>L2</td>
<td>L3</td>
<td>L4</td>
<td>L5</td>
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<td>[kNm]</td>
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<td></td>
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<tr>
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Table 6.5 Direct comparison of sagging moment envelopes based on FBG measurement data and based on benchmark grillage model.

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</table>
### 6.3 Comparison of monitoring data with model predictions

Table 6.6 Sensitivity analysis summary for the Chebsey bridge grillage analysis.

<table>
<thead>
<tr>
<th>Model parameter or modelling assumption</th>
<th>Benchmark model</th>
<th>Sensitivity analysis</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete modulus of elasticity, $E_c$</td>
<td>C60/75 value (i.e., 39.1 MPa)</td>
<td>• C40/50 value (i.e., 35.2 MPa) • C90/105 value (i.e., 43.6 MPa)</td>
<td>Uniform change of $E$ across the deck has no effect on load distribution and bending moment (i.e., load effect) under simply supported or fully fixed boundary conditions and has little effect under partially fixed boundary conditions</td>
</tr>
<tr>
<td>Transverse grillage beam depth, $d_{transverse}$</td>
<td>In-situ concrete depth (i.e., 765 mm)</td>
<td>• $d = 900$ mm (full depth of the deck) • $d = 1800$ mm (arbitrary value)</td>
<td>Little effect on load distribution and bending moment (e.g., for $d = 1800$ mm, $&lt; 1.0$ kNm difference at mid-span and $&lt; 2.0$ kNm difference at quarter-spans)</td>
</tr>
<tr>
<td>Torsional rigidity, $J$</td>
<td>Full torsion (i.e., 100% $J$)</td>
<td>No torsion (i.e., $J = 0$)</td>
<td>Little effect on load distribution and bending moment ($&lt; 0.5$ kNm difference at mid-span and $&lt; 1.5$ kNm difference at quarter-spans)</td>
</tr>
<tr>
<td>Longitudinal grillage beam depth for edge beams, $d_{longitudinal,TYE7}$</td>
<td>Sum of slab and girder depths (i.e., 900 mm)</td>
<td>$d_{longitudinal,TYE7} = 1200$ mm (arbitrary value; due to partial composite action between edge beam and parapet – refer to section 6.2.2)</td>
<td>Large effect on load distribution and bending moment (e.g., reduction of around 10 kNm for BM5 mid-span) due to edge beams (partially bonded to the parapets) attracting more loads</td>
</tr>
</tbody>
</table>

Continued on next page
### Table 6.6 – continued from previous page

<table>
<thead>
<tr>
<th>Model parameter or modelling assumption</th>
<th>Benchmark model</th>
<th>Sensitivity analysis</th>
<th>Comments</th>
</tr>
</thead>
</table>
| Support boundary conditions – rotational spring stiffness, $K_r$ | Simply supported (i.e., $K_r = 0$) | • Fully fixed (i.e., $K_r = \infty$)  
• Partially fixed (e.g., $K_r = 50, 100, 500 \text{ MNm/rad}$) | Large effect on bending moment (e.g., 61% reduction for BM5 mid-span under fully fixed boundary conditions); large effect on load distribution (especially at locations near end supports) |
| Vertical distribution of axle loads from rails to deck (through ballast) | Applied based on EN 1991-2:2003 (i.e., ballast-to-width ratio of 4:1; could also refer to Lin et al. (2019) for more details) | • Not applied (i.e., as direct point loads)  
• Ballast-to-width ratio (i.e., the ratio between ballast depth and load patch width) of 1:1 | Little effect on load distribution and bending moment (< 1.0 kNm difference across the deck) |
| Transverse distribution of axle loads due to the effect of canted ballast track (refer to section 4.2) | Applied based on EN 1991-2:2003 (i.e., in this case 59%/41% for the two rails) | Not applied (i.e., 50%/50% for the two rails) | Very small effect on load distribution and bending moment (< 1.5 kNm difference across the deck) |
Table 6.6 – continued from previous page

<table>
<thead>
<tr>
<th>Model parameter or modelling assumption</th>
<th>Benchmark model</th>
<th>Sensitivity analysis</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modelling method</td>
<td>Benchmark grillage model (refer to section 6.3.1)</td>
<td>• Changing grillage mesh size (e.g., double the longitudinal grillage beam width, double the transverse grillage beam width) • FE model using shell elements</td>
<td>Negligible effect on model predictions of bending moment</td>
</tr>
</tbody>
</table>

6.3.3 Structural model updating: updating boundary conditions and stiffness contribution of secondary elements

Boundary conditions were first examined in the model updating process, which directly affects the longitudinal bending moment profile. For example, partially fixed boundary conditions can cause hogging moments at locations close to end supports during a train-passage event. In this study, to examine the support boundary conditions, the longitudinal profile of bending curvature for each beam was first normalised relative to the mid-span curvature, using Equation 6.6:

$$\kappa_{\text{normalised relative to mid-span,BMi}}(x) = \frac{\kappa_{\text{BMi}}(x)}{\kappa_{\text{BMi}}(x = \text{mid-span})}$$  \hspace{1cm} (6.6)

where $x$ is the longitudinal location along each beam and $i$ is the beam number ($i = 1, 2, 3, 4, 5, 9$). This normalised curvature was used to update support boundary conditions. This is because this normalised curvature profile is found to be highly sensitive to boundary conditions, and highly insensitive to modulus of elasticity ($E_c$) and the magnitude of input axle loads.
The rotational spring stiffness $K_r$ was estimated and updated by matching the longitudinal normalised curvature envelope (relative to mid-span) predicted from the grillage model against that derived from the FBG data during a train-passage event. Figure 6.9a shows the longitudinal normalised curvature envelope (relative to mid-span) for the six instrumented girders based on the FBG data under Train No. 3. Note that no curvature data is available at L10 (i.e., $x = 10450$ mm) because data from only one sensor was available for each beam at this location (refer to Table 6.3 note). It can be seen that there is a general agreement in longitudinal normalised curvature envelope across the six instrumented girders, although there are also some random variations across these girders. Figure 6.9b shows the mean and variation ($\pm$1 standard deviation) of the longitudinal normalised curvature envelopes across the six instrumented girders.

The longitudinal normalised curvature envelope derived from FBG measurements was then compared with the corresponding predictions from a series of grillage models with different values of rotational spring stiffness, $K_r$. Specifically, $K_r = 0$ (i.e., simply supported), 100 MNm/rad, 200 MNm/rad, 300 MNm/rad, 400 MNm/rad, 500 MNm/rad and infinity (i.e., fully fixed). It should be noted that the rotational spring stiffness of bridge elastomeric bearing pads (similar to the ones used in the Chebsey bridge, in terms of dimensions and material) is typically of the order of 10-50 MNm/rad (Akogul and Celik, 2008; Vidot-Vega et al., 2009).

First, it was assumed in these models that both supports have equal $K_r$ (i.e., $K_{r1} = K_{r2}$). The results are plotted in Figure 6.10. It can be seen from this figure that the data clearly shows that the bridge deck is not simply supported, although there is a relatively large degree of variation in the normalised curvature profile. The deck supports are found to be partially fixed with some degree of rotational restraint. Under the assumption of equal $K_r$ for both supports, using manual tuning (refer to section 2.2.5 for explanation), $K_{r1} = K_{r2} = 200$ MNm/rad gives a good match between model prediction and sensor measurement in terms of normalised bending curvature relative to mid-span.

In addition, it can be seen from Figure 6.10 that the measured normalised curvature profile was not symmetrical, suggesting that the two end supports have different degrees of partial fixity (i.e., $K_{r1} \neq K_{r2}$). It appears that the east support (LHS in Figure 6.10) has a higher degree of partial fixity than the west support (RHS in Figure 6.10), i.e., $K_{r1} > K_{r2}$. Using additional manual tuning, $K_{r1} = 250$ MNm/rad and $K_{r2} = 150$ MNm/rad gives a better match between model prediction and sensor measurement in terms of normalised bending curvature relative to mid-span.
6.3 Comparison of monitoring data with model predictions

Fig. 6.9 Normalised curvature envelope (relative to mid-span) based on FBG measurement data: (a) for the six instrumented girders; and (b) showing mean and variation (±1 standard deviation) across the six instrumented girders.
6.3 Comparison of monitoring data with model predictions

Fig. 6.10 Normalised curvature envelope for structural grillage models with different rotational stiffness (assuming equal $K_r$ for both supports in the models) compared to curvature derived from measured sensor data.

The contribution of secondary elements (e.g., bridge parapets above edge beams) can be inferred from the measured neutral axis positions. As shown previously in section 6.2.2 and Figure 6.2, the edge TYE7 beams have distinctively higher neutral axis levels than the internal TY7 beams. The larger cross-section for the edge composite beam section (as a result of the partial composite action between the parapets and the bridge deck) would attract more moment than the internal beams which do not have parapets attached. It is difficult to directly relate the neutral axis position ($z_{N,A}$) to the cross-section second moment of area ($I$) which in turn affects the bending stiffness ($EI$). A conservative assumption was made by using the smallest possible second moment of area which gives the measured neutral axis level. This was achieved by assuming that all of the additional cross-sectional area due to the partially bonded secondary element (e.g., bridge parapet) was located at the top surface of the deck. Using this assumption, it was found that the updated $I$ value would be 8% larger for BM1 and 3% larger for BM9. The effects of these differences on model predictions were found to be very small in the analysis, and therefore the contribution of secondary elements is not a significant contributing factor to the model-data discrepancy in this study. This confirms the previous conclusion of negligible secondary element contribution (made in section 6.2.2).
6.3.4 Model-data comparison and interpreting model-data discrepancy: moment envelope and load distribution factor

Figure 6.11 provides a summary of the model-data comparison of the moment envelope results. The error bars show the variation (±1 standard deviation) in bending moment across different girders at each longitudinal location. Overall, Figure 6.11 shows that the partially fixed boundary conditions contribute a significant part of the discrepancy between the benchmark model predictions and sensor measurements. By updating the boundary conditions, the moment envelope profile from the updated model is significantly closer to that based on the sensor data.

It should also be noted that simplified assumptions have been made in the structural modelling and analysis, such as uniform $E$ across the deck, uniform boundary conditions (rotational spring stiffness, $K_r$) across different beam locations at each end support, and uniform contribution of secondary element (e.g., parapet) along different longitudinal locations for each edge beam. In reality, there are likely to be local variations in these properties, resulting in local variations in structural response. For example, it can be seen from Table 6.3 and Figure 6.9 that although the data shows an overall uniform load sharing across different beams, there are some local “anomalies” such as BM1 L1 and BM2 L1, which have significantly higher bending moment than the rest of the instrumented girders at L1. This could be due to local variations in the end support rotational restraint or in the degree of partial composite action between the beams and the secondary elements near these locations.

As for transverse load distribution, Figure 6.12 provides a summary of the model-data comparison of the load distribution factor (DF) results. Both the grillage models and the sensor data show a near uniform transverse load distribution across different girders. The DF values based on the grillage model predictions and based on the sensor data are distinctively lower than the typical design DF value based on Morice and Little (1955), suggesting that the latter is conservative. This means that the bridge deck is very stiff transversely and hence acts predominantly as a rigid beam. This results in near uniform transverse load distribution and therefore the bridge is able to carry more load than that predicted using the equations in Morice and Little (1955).
6.3 Comparison of monitoring data with model predictions

Fig. 6.11 Model-data comparison of moment envelope (error bars showing variation, ±1 standard deviation, across different beams).

Fig. 6.12 Model-data comparison of load distribution factor (based on the average of L5 and L6 moment) – left to right: BM9, BM5 to BM1.
6.4 Discussion

Compared with the existing body of literature on monitoring structural behaviour under live load, this study provides a novel methodology of using “normalised” real-time curvature profiles to evaluate load distribution characteristics and to calibrate support boundary conditions, which cover two important modelling assumptions about bridge behaviour in bridge structural assessment. The methodology in this study works under operational conditions, and it is better than the conventional methodology of using controlled load testing in several aspects: (i) no bridge closure is needed; (ii) more data points can be collected over time to minimise uncertainty and improve reliability of the results; and (iii) the effects of uncertain material stiffness and load magnitude on model updating, particularly the calibration of support boundary conditions, can be minimised as a result of using normalised curvature profiles.

The results presented in this study have demonstrated the feasibility of monitoring and evaluating commonly adopted modelling assumptions about bridge behaviour under live load, and in particular, contribution of secondary elements, load distribution characteristics and support boundary conditions, to enable more realistic capacity assessment of girder bridges. The methodology in this study can be applied to many types of girder bridges (e.g., prestressed concrete girder, steel girder, steel-composite girder).

6.5 Summary

This chapter has evaluated the in-service structural behaviour of the Chebsey bridge under live load (specifically, six train-passage events on 27th September 2017). The study has demonstrated how strain monitoring data may be used to better understand and quantify certain potentially conservative assumptions about in-service structural behaviour and enable more realistic structural modelling and analysis (e.g., by structural model updating) for bridge assessment purposes. Specifically, these assumptions include the contribution of secondary elements (e.g., assumed as zero contribution), load distribution effects (e.g., using load distribution factor values in codes and standards) and boundary conditions (e.g., assumed as simply supported for bridges with simple supports). A summary of the key findings is listed below:

- Neutral axis positions at measured cross-sections along each girder can be obtained from the top and bottom FBG sensor measurements of strain at the top and bottom prestressing strands. This can then be used to infer the degree of partial composite action between the solid slab bridge deck and the secondary elements (e.g., parapets,
6.5 Summary

surfacing layer) and thus infer the contribution of these secondary elements to structural stiffness.

- For the Chebsey bridge, it has been found that the edge girders have distinctively higher neutral axis positions than the internal girders (around 40 mm higher for edge girder BM1 and around 15 mm higher for edge girder BM9), indicating that the parapets are partially bonded to the bridge deck. However, the degree of partial composite action between the bridge deck and the parapets is found to be very small. In addition, it has been inferred from the measured neutral axis positions that the surfacing layer has negligible stiffness contribution to the bridge deck. Overall, the stiffness contribution of secondary elements such as parapets is insignificant for the Chebsey bridge and may be discarded in the structural assessment of this bridge.

- Bending curvature and moment can be estimated from the top and bottom sensor measurements of strain, assuming a linear strain profile at each cross-section. To investigate the transverse load distribution across different girders under live load, the curvature or moment profiles of different girders have been normalised relative to one girder (e.g., the central girder) in real time to enable comparison between different girders. Load distribution factors (DF) have been computed to quantify the degree of transverse load sharing.

- For the Chebsey bridge, it has been found that there is near uniform transverse load distribution across the nine girders and the amount of transverse bending of the bridge deck is extremely small. This suggests that the bridge deck acts predominantly as a very stiff single beam in longitudinal bending with very little transverse bending. The measured DF is distinctively smaller than the typical design DF based on Morice and Little (1955) (smaller by approximately 10%).

- Based on a comprehensive sensitivity analysis, the sources of uncertainty which would have a large effect on the discrepancy between model predicted and sensor measured load effects (e.g., bending moment) have been identified: (i) support boundary conditions (e.g., rotational spring stiffness, $K_r$), (ii) input axle loads, (iii) concrete modulus of elasticity ($E_c$), and (iv) contribution of secondary elements (affecting the second moment of area, $I$). This suggests that these sources of uncertainty require careful examination when validating and updating the structural model.

- The normalised curvature envelopes of the instrumented girders under a train-passage event (Train No. 3) have been used to update the rotational spring stiffness ($K_r$) of the two end supports. This is because the normalised curvature envelope of a girder is
highly insensitive to the concrete modulus of elasticity and the magnitude of input axle loads, and is primarily sensitive to the support boundary conditions.

• For the Chebsey bridge, it has been found from structural model updating that the boundary conditions are partially fixed (as distinct to the design assumption of simply supported boundary conditions) and this is the main reason behind the model-data discrepancy in this study (specifically, the model predicted live load effects are significantly larger than the measured live load effects).
Chapter 7

Structural Utilisation

7.1 Introduction

This chapter investigates how monitoring data can be utilised to evaluate and visualise structural utilisation during bridge operation and how this information can be used to inform the “margin of capacity”, which is defined in this study as how much additional live load can be safely placed on a bridge without violating the design performance criteria. This is important for understanding the efficiency of bridge asset utilisation and facilitating the optimisation of traffic management (e.g., load and route combination) and more targeted bridge maintenance (e.g., focus on structural elements which are highly utilised and critical to structural integrity). This can also provide new insights into the conservativeness of the original design.

In this study, three definitions of structural utilisation have been proposed (specifically, live load effect utilisation, live load capacity utilisation and total capacity utilisation). These multiple definitions are different in their engineering interpretation, level of complexity, use case and degree of conservativeness for informing the “margin of capacity”. These different types of structural utilisation are then evaluated for the Chebsey bridge using the collected monitoring data. Finally, three different types of visualisation for structural utilisation are presented.

7.2 Definitions of structural utilisation

In this study, “margin of capacity” is defined as how much additional live load can be safely placed on a bridge without violating the design performance criteria. To facilitate the investigation of “margin of capacity”, three definitions of structural utilisation have
been proposed: live load effect utilisation, live load capacity utilisation and total capacity utilisation. These are described and explained in more detail in the three subsections below (7.2.1, 7.2.2 and 7.2.3).

7.2.1 Live load effect utilisation

*Live load effect utilisation* is defined as the ratio between the actual live load effect on a structure (e.g., derived from sensor measurements of strain and material testing results) and the design live load effect for this structure (e.g., at SLS or ULS), as shown in Equation 7.1:

$$\text{Live load effect utilisation} = \frac{\text{Actual live load effect}}{\text{Design live load effect}}$$ (7.1)

This definition can be used to assess the degree of conservativeness in the design live load effect. Note that this definition considers live load only and does not consider other load cases or resistance (i.e., load capacity). In terms of using this definition to inform the “margin of capacity”, it is based on the premise that if the measured live load effect is smaller than or equal to the design live load effect (in other words, the allowable live load effect in design), the bridge may still be deemed safe and performing satisfactorily.

Live load effect utilisation is most often smaller than 1 because of two reasons: the use of load factors to account for uncertainty, and the conservative modelling assumptions made in the design analysis model for predicting the live load effects on the bridge. In the Eurocode, load factors include partial safety factors on load ($\gamma_f$), load combination factors ($\psi_0$, $\psi_1$, $\psi_2$), dynamic factors (e.g., $\phi$) and additional factors to adjust for heavier or lighter traffic load compared to normal traffic (e.g., $\alpha$). As for the conservative modelling assumptions used for predicting the live load effect, these include: (i) the adopted live load model (e.g., characteristic values of live load actions in design codes, bridge specific assessment live load (BSALL) based on weigh-in-motion data), (ii) the assumed boundary conditions (which affect the predicted live load effect such as bending moment), and (iii) the assumed effects of load distribution (e.g., transverse and vertical load distribution through the track ballast and across the bridge deck).

7.2.2 Live load capacity utilisation

*Live load capacity utilisation* is defined as the ratio between the actual live load effect on a structure (e.g., derived from sensor measurements of strain and material testing results) and the actual or best estimate live load capacity of this structure (e.g., minimum live load effect that would cause the bridge to violate its design performance criteria such as SLS or ULS,
which could be derived based on design calculations or estimated based on material testing results, load testing results and/or strain monitoring data), as shown in Equation 7.2:

\[
\text{Live load capacity utilisation} = \frac{\text{Actual live load effect}}{\text{Actual or best estimate live load capacity}} \tag{7.2}
\]

It should be noted that the actual or best estimate live load capacity (i.e., the denominator in Equation 7.2) is most likely not equal to the design live load effect (i.e., the denominator in Equation 7.1). This is because the former can also include additional live load capacity that could be unlocked by the extent of overdesign (e.g., how much the design resistance \(R_d\) is greater than the design load effect \(E_d\) for the governing design criterion such as SLS stress, ULS moment or ULS shear) (Orr et al., 2019), any conservativeness in the predicted effects of other load actions (e.g., dead load, super-imposed dead load, environmental loads) and any conservativeness in resistance prediction (e.g., predicted level of prestress loss for prestressed concrete structures, which affects load capacity).

For the evaluation of live load capacity utilisation, strain monitoring data can mainly be used to evaluate the actual live load effect (as demonstrated in Chapter 6), although it can also be used to better estimate the live load capacity of prestressed concrete bridges through better estimating the level of prestress in the prestressed concrete beams (as demonstrated in Chapter 5).

### 7.2.3 Total capacity utilisation

*Total capacity utilisation* is defined as the ratio between the actual or best estimate total load effect (i.e., taking into account all load actions such as dead load, super-imposed dead load, live load and environmental loads) and the actual or best estimate total load capacity (i.e., resistance), as shown in Equation 7.3. Where no measurement of a certain load effect or total load capacity is available, the design value may be used as an alternative.

\[
\text{Total capacity utilisation} = \frac{\text{Actual or best estimate total load effect}}{\text{Actual or best estimate total load capacity}} \tag{7.3}
\]

As for prestressed concrete bridges, in design, the SLS stress criterion is most often the governing design criterion. As an example, Equation 7.4 shows the more specific definition
of total capacity utilisation (under SLS stress criterion) for a prestressed concrete structure:

\[
\text{Total capacity utilisation (under SLS stress criterion for prestressed concrete structure)} = \frac{\text{Actual or best estimate total stress due to all loads}}{\text{Actual or best estimate total stress capacity (SLS)}} = \frac{\text{Actual or best estimate } (\sigma_{DL} + \sigma_{SDL} + \sigma_{LL} + \sigma_{\text{environment}})}{\text{Actual or best estimate } (\sigma_{\text{prestress}} - \sigma_{\text{limit, SLS}})} \tag{7.4}
\]

where \(\sigma_{DL} + \sigma_{SDL} + \sigma_{LL} + \sigma_{\text{environment}}\) is the total stress at the location of interest due to the combination of dead load, super-imposed dead load, live load and environmental loads; \(\sigma_{\text{prestress}}\) is the stress at the location of interest due to the prestressing force; and \(\sigma_{\text{limit, SLS}}\) is the SLS stress limit of interest (e.g., tension limit or compression limit; at transfer of prestress or at working load). The denominator of Equation 7.4, \(\sigma_{\text{prestress}} - \sigma_{\text{limit, SLS}}\), can be considered as the total stress capacity at the location of interest of the prestressed concrete structure. The numerator of Equation 7.4, \(\sigma_{DL} + \sigma_{SDL} + \sigma_{LL} + \sigma_{\text{environment}}\), can be considered as the consumed or utilised stress capacity.

For the evaluation of total capacity utilisation, strain monitoring data can be used to better estimate certain load effects (which will be discussed in section 7.3.1), although it can also be used to better estimate the live load capacity of prestressed concrete bridges through better estimating the level of prestress, \(\sigma_{\text{prestress}}\), in the prestressed concrete beams (as demonstrated in Chapter 5).

### 7.3 Evaluation of structural utilisation

#### 7.3.1 Strain measurements to determine actual load effects

This section explains how strain monitoring data can be utilised to measure different load effects and thus provide more accurate evaluation of structural utilisation (as defined previously in section 7.2) in bridge assessment. Table 7.1 provides a summary of discussions on how strain measurements can be utilised for evaluating actual load effects in prestressed concrete bridges (specifically, stresses due to different load actions and the prestressing force).
Table 7.1 Discussions on how strain measurements can be utilised to determine actual load effects in prestressed concrete bridges.

<table>
<thead>
<tr>
<th>Load case</th>
<th>Discussion on the corresponding strain measurement (if there is any) for load effect</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prestress</td>
<td>Prestress loss may be estimated from the change of permanent strain (i.e., static strain) over time at the centroidal level of each girder, which may be evaluated by linear interpolation of measured changes of permanent strain at the top and bottom prestressing tendon in each girder.</td>
</tr>
</tbody>
</table>

Continued on next page
### Table 7.1 – continued from previous page

<table>
<thead>
<tr>
<th>Load action</th>
<th>Discussion on the corresponding strain measurement (if there is any) for load effect</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dead load (DL), superimposed dead load (SDL) and live load (LL)</td>
<td></td>
</tr>
</tbody>
</table>
- Only change of strain (from a certain starting point) can be measured rather than absolute strain. Therefore, continuous strain measurement during each relevant construction or operational stage may be needed in order to capture the load effects of different components or events. For example, for DL, relevant stages include beam installation and concrete deck casting; for SDL, relevant stages include the installation of different secondary elements such as parapets, robust kerbs, surfacing layers, ballast and track; and for LL, relevant stages include different train-passage events.  
- Measured changes of strain at the top and bottom sensors of each cross-section can be used to evaluate changes of curvature at the cross-section, using $\Delta \kappa = \frac{\Delta \varepsilon_{\text{top}} - \Delta \varepsilon_{\text{bottom}}}{z_{\text{top}} - z_{\text{bottom}}}$ (refer to Equation 6.2).  
- Notes:  
1. For the Chebsey bridge: no such continuous strain measurement was taken for DL or SDL (refer to Table 4.1).  
2. As a result of prestressing, particularly at the early age of the structure, there is also the effect of continued beam camber, which results in upward deflection and bending (i.e., hogging). This is why continuous measurement at each construction stage for DL would be needed in order to decouple the effects of DL and beam camber.  
3. For the Chebsey bridge, unpropped construction was used for the composite construction (precast beams and in-situ concrete infill deck slab). Therefore, DL may be deemed as being carried by the precast section (with simply supported boundary conditions) and additional SDL and LL may be deemed as being carried by the finished composite section (with partially fixed boundary conditions due to the use of concrete infill behind the girders).  

Continued on next page
7.3 Evaluation of structural utilisation

Table 7.1 – continued from previous page

<table>
<thead>
<tr>
<th>Load action</th>
<th>Discussion on the corresponding strain measurement (if there is any) for load effect</th>
</tr>
</thead>
<tbody>
<tr>
<td>Environmental load (e.g., thermal load, wind load)</td>
<td>• Long-term strain measurement at regular intervals during construction and operation may facilitate the more realistic characterisation and evaluation of environmental effects (e.g., diurnal, seasonal), such as thermal strain.</td>
</tr>
<tr>
<td></td>
<td>• Notes:</td>
</tr>
<tr>
<td></td>
<td>1. For the Chebsey bridge: no such long-term strain measurement at regular intervals was taken (refer to Table 4.1).</td>
</tr>
<tr>
<td></td>
<td>2. For the Chebsey bridge: the effect of wind load was found to be negligible in design compared with the effects of other load actions, and hence no attempt to measure wind speed and direction was made.</td>
</tr>
</tbody>
</table>

Information on the sensor data collected for the Chebsey bridge at different construction and operational stages was previously summarised in Table 4.1 in Chapter 4. Based on Table 7.1 and Table 4.1, for the Chebsey bridge, the strain monitoring data collected can be used to provide a more realistic evaluation of two load effects: (i) prestress (refer to Chapter 5), and (ii) live load response (refer to Chapter 6). In addition, the load effect of super-imposed dead load predicted in the design could potentially be modified using the partially fixed boundary conditions calibrated using the monitoring data (refer to section 6.3.3), assuming that the partially fixed boundary conditions fully developed before the application of super-imposed dead load (e.g., installations of parapets, robust kerbs, surfacing layers, ballast, track).

As mentioned in section 4.2, the design of the Chebsey bridge was governed by the SLS check. Specifically, the stress profiles across the bridge deck were checked against the serviceability stress limits both during construction and in operation. The following subsection (7.3.2) presents the evaluation of structural utilisation against the SLS stress design criterion. All three definitions of structural utilisation presented in section 7.2 were considered (i.e., live load effect utilisation, live load capacity utilisation, and total capacity utilisation).
7.3.2 Utilisation against SLS stress criterion (for prestressed concrete bridges)

To demonstrate how structural utilisation was calculated based on the three definitions in section 7.2 (live load effect utilisation, live load capacity utilisation, and total capacity utilisation), an example of the stresses at the beam soffit of BM5 (the central beam) at mid-span is presented, as shown in Table 7.2. This was expected to be the most critical location under the SLS stress design criterion. Similar calculations were performed for other vertical locations through the depth of the bridge deck (e.g., beam top, deck top), other locations along the length of the beam (e.g., near support, quarter span) and other beams across the deck.

It can be seen from Table 7.2 that three types of stress could be modified using sensor data, which are prestress (direct measurement of prestress loss), stress under train loads (derived from measurements of dynamic strain under train-passage events) and stress under superimposed dead load (using modified predictions based on calibrated boundary conditions). Based on the modifications using the sensor data (where no measurement is available, SLS frequent design value is used as an alternative), the beam soffit of BM5 mid-span experiences a maximum compressive stress of 5.2 MPa. This means that an additional load effect of 5.2 MPa tensile stress may be added before violating the SLS criterion (specifically, no tensile stress at beam soffit under the SLS frequent load combination). Based on the measurement, the heaviest train measured in this study can cause a tensile stress of 0.9 MPa at the beam soffit of BM5 mid-span. The live load effect utilisation for this particular train-passage event is 26%, which means that additional 2.8 identical trains may be placed on the bridge before the actual live load effect reaches the design live load effect. The live load capacity utilisation (SLS stress) for this particular train-passage event is 15%, which means that additional 5.6 identical trains may be placed on the bridge before violating the design SLS stress criterion.

7.4 Visualisation of structural utilisation

7.4.1 Overview

This section investigates different ways of presenting the data and information of structural utilisation to bridge owners and bridge engineers in order to: (i) enable more direct and intuitive interpretation of the data and information on structural utilisation and “margin of capacity” (e.g., inform how much additional live load can be safely placed on a bridge without violating the design performance criteria); and (ii) support decision making in bridge
Table 7.2 Structural utilisation against SLS stress criterion, at the soffit of beam BM5 at mid-span.

<table>
<thead>
<tr>
<th>Load case</th>
<th>Stress at BM5 soffit at mid-span [MPa] (+ve is compressive and –ve is tensile)</th>
<th>Measured load effect Design load effect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Load effect using characteristic or nominal value</td>
<td>Design load effect (using Eurocode SLS frequent load combination)</td>
</tr>
<tr>
<td>Prestress</td>
<td>14.6 (with 19.6% pre-stress loss)</td>
<td>14.6</td>
</tr>
<tr>
<td>Differential shrinkage</td>
<td>-0.1</td>
<td>-0.1</td>
</tr>
<tr>
<td>Dead load</td>
<td>-5.5</td>
<td>-5.5</td>
</tr>
<tr>
<td>Super-imposed dead load (including ballast)</td>
<td>-2.0</td>
<td>-2.0</td>
</tr>
<tr>
<td>Train load (under EC1 LM71)</td>
<td>-2.5</td>
<td>-3.4</td>
</tr>
<tr>
<td>Thermal load</td>
<td>-2.1</td>
<td>-1.1</td>
</tr>
<tr>
<td>Sum of load effects</td>
<td>N/A</td>
<td>2.6</td>
</tr>
<tr>
<td>Stress limit</td>
<td>0.0</td>
<td>N/A</td>
</tr>
<tr>
<td>Difference</td>
<td>-2.6</td>
<td>-5.2</td>
</tr>
<tr>
<td>Live load capacity utilisation (SLS stress)</td>
<td>N/A</td>
<td>57% ($3.4 + 2.5 \times 100%$)</td>
</tr>
<tr>
<td>Total capacity utilisation (SLS stress)</td>
<td>82% ($14.6 - 2.6 \times 100%$)</td>
<td>66% ($15.2 - 5.2 \times 100%$)</td>
</tr>
</tbody>
</table>

1 Note: Characteristic or nominal values of material and geometric properties are used where appropriate in the calculations, based on the Eurocode equations and provisions.

2 Note: Design prediction is based on the assumption of simply supported boundary conditions. Super-imposed dead load is carried by the finished composite section (with partially fixed boundary conditions due to the use of concrete infill behind the girders). The prediction is modified based on the calibrated support boundary conditions using structural model updating (refer to section 6.3.3 on calibrating boundary conditions).

3 Note: $-3.4 = \psi_1 \times \alpha \times \Phi_3 \times -2.5 = 0.8 \times 1.21 \times 1.39 \times -2.5$, where $\psi_1$ is the load combination factor, $\alpha$ is the additional load factor applied on LM71, and $\Phi_3$ is the dynamic factor for track with standard maintenance.

4 Note: The results from heating, which include both uniform and differential heating, are presented as in this case it is more critical than cooling for beam soffit.

5 Note: Where no measurement is available, design value (SLS frequent) is used as an alternative.
O&M (e.g., inform which parts of the bridge are more critical and vulnerable and thus require careful examination and possible interventions).

Three types of visualisation were considered:

1. **Bar plot**: This is used for direct visual comparison between measured load effects and design load effects, and in some cases, detailed breakdown of structural utilisation (i.e., contributions from different load actions). The objective is to enable bridge engineers to better analyse the numerical results of load effects and structural utilisation and directly visualise the “margin of capacity”.

2. **“Live” utilisation heatmap**: This provides a global visualisation of utilisation across the whole structure at a particular timestamp or during a particular event (e.g., envelope of results during a train-passage event). The objective is to enable bridge engineers to better visualise and understand the load distribution, and in particular, which parts of the structure are highly utilised and which parts are not.

3. **Spatiotemporal utilisation heatmap**: This provides a historic summary of past utilisation in one plot as it is difficult to visualise numerous “live” utilisation heatmaps simultaneously from numerous timestamps in the past. For example, it may be used to visualise the overall utilisation state over time and the proportion of “overload” events (e.g., the proportion of time when utilisation exceeded a certain pre-defined value).

The following three subsections (7.4.2 to 7.4.4) present the above-mentioned three types of plots for visualising structural utilisation, using the three utilisation definitions in section 7.2 (live load effect utilisation, live load capacity utilisation, and total capacity utilisation).

### 7.4.2 Bar plot

Figure 7.1 shows a live load effect utilisation bar plot for BM5 at mid-span under the six train-passage events measured. Trains No. 3 and No. 4 are passenger trains, and the rest are freight trains. Note that the difference between the characteristic LM71 load effect (where LM71 is the load model used in design for train loads) and the SLS frequent LM71 load effect is due to the latter having an additional load factor applied on LM71 for traffic load adjustment ($\alpha = 1.21$) and a dynamic factor applied for track with standard maintenance ($\Phi_3 = 1.39$).

Figure 7.1 can be used to directly visualise the live load effect utilisation, and in particular, the difference between the measured live load effect and the calculated design live load effect (e.g., SLS frequent, ULS Set B, as specified in the figure). For example, it can be seen that
the live load effect under the heaviest train measured (Train No. 6) is below 25% of the SLS frequent design live load effect and below 40% of the characteristic LM71 load effect. This suggests that the design live load effect may be highly conservative, which could be due to a combination of reasons such as a conservative live load model (LM71), conservative load factors (e.g., γ, α, Φ₃) and conservative modelling assumptions used in the analysis such as boundary conditions (e.g., assumed as simply supported).

Figure 7.2 shows a combined live load effect utilisation and live load capacity utilisation bar plot for BM5 mid-span under the six train-passage events measured. The measured live load effect is compared against the following live load effect or live load capacity:

**Live load effect**

1. Characteristic live load (LM71) effect
2. SLS frequent design live load (LM71) effect
3. ULS Set B design live load (LM71) effect

**Live load capacity**

4. SLS frequent live load capacity based on the design model
5. ULS Set B live load capacity based on the design model

6. Modified SLS frequent live load capacity based on the monitoring data

7. Modified ULS Set B live load capacity based on the monitoring data

(4) and (5) take into account additional live load capacity unlocked by the extent of overdesign, and (6) and (7) take into account additional live load capacity unlocked by both overdesign and any relaxed conservative design assumptions modified using the monitoring data (e.g., boundary conditions).

Figure 7.2 can be used to visualise the live load capacity utilisation, and in particular, the difference between the measured live load effect and the design or best estimate live load capacity. The difference between the design live load effect and the design live load capacity indicates the degree of conservativeness in the original design. The difference between the design live load capacity and the modified live load capacity based on monitoring data shows the effect of relaxing conservative modelling assumptions (in this case, boundary conditions) based on the collected monitoring data.

Fig. 7.2 Live load effect utilisation and live load capacity utilisation bar plot: BM5 mid-span.

Figure 7.3 shows a total capacity utilisation bar plot for SLS stress at the soffit of BM5 (central beam) under the heaviest train measured (Train No. 6). Envelope live load results, i.e., maximum measured live load effect at each sensor location during the train-passage
event, are used. Where no measurement is available, the predicted design load effect is used as an alternative.

Figure 7.3 can be used to visualise directly:

1. The total stress capacity (SLS), i.e., the difference between the level of precompression due to prestress and the stress limit of interest (tensile or compressive)

2. The utilised stress capacity (SLS) and the contribution of each load action (or load effect) to the utilised stress capacity

3. The total capacity utilisation (SLS stress), i.e., the ratio of the utilised stress capacity (SLS) to the total stress capacity (SLS)

4. The residual stress capacity (SLS), i.e., the difference between the actual measured stress state and the design stress limit of interest (tensile or compressive)

5. How much additional live load may be added before violating the tensile stress limit (i.e., SLS)

For example, it can be seen from Figure 7.3 that near the mid-span location of BM5 beam soffit, permanent loads (DL + SDL) utilised approximately 50% of the total tensile stress capacity (i.e., the difference between the level of precompression due to prestress and the tensile stress limit). Live load only utilised a very small proportion of the total tensile stress capacity. In addition, the existing live load effect is only a very small proportion of the residual tensile stress capacity, which suggests that much more additional live load may be added before the stress state reaches the tensile stress limit (i.e., violating the SLS).

7.4.3 “Live” utilisation heatmap

Figure 7.4 shows two live load effect utilisation heatmaps (for SLS frequent and characteristic load effects (LM71), respectively) for the bridge deck under the heaviest train measured (Train No. 6). Envelope results, i.e., maximum live load effect utilisation at each location over the plan area of the bridge deck slab during the train-passage event, are shown. For live load response, only discrete measurement data (i.e., at specific point locations) using FBG sensors were taken. Each rectangle represents the live load effect utilisation at the centre location of that rectangle (where the FBG sensor is located). Note that no sensor data was available for BM6 to BM8 and for near support locations of all the beams, and therefore no results are shown for these locations. Overall, it can be visualised from the heatmaps that the live load effect utilisation was low across the whole deck under this train-passage event.
7.4 Visualisation of structural utilisation

Fig. 7.3 Total capacity utilisation (SLS stress) bar plot: BM5 beam soffit. Note: (1) for traffic load, envelope results under Train No. 6 (the heaviest train measured) are used; and (2) where no measurement is available (specifically, dead load, super-imposed dead load and thermal load), design load effect (SLS frequent) is used as an alternative.
and the live load distribution was near uniform across the different beams. In addition, the maximum live load effect utilisation occurred at slightly to the east (RHS direction in the figure) of the mid-span locations.

Figure 7.5 shows a live load capacity utilisation heatmap based on the SLS stress criterion, under the heaviest train measured (Train No. 6). It can be visualised from the heatmap that the live load capacity utilisation was extremely low across the whole deck under this train-passage event.

Figure 7.6 shows a total capacity utilisation (SLS stress) heatmap for the bridge deck under the heaviest train measured (Train No. 6). It can be seen from Figure 7.6 that the total capacity utilisation (SLS stress) was relatively high at the central region of the bridge deck (i.e., near mid-span, near central beam) and low at other locations, which is as expected.

### 7.4.4 Spatiotemporal utilisation heatmap

Figure 7.7 shows an example of a spatiotemporal live load effect utilisation heatmap for the bridge deck, using the six train-passage events measured. Envelope results, i.e., the maximum live load effect utilisation at each location (mid-span of each beam) during each train-passage event, are shown. With more data collected for more train-passage events in the future, such a spatiotemporal live load effect utilisation heatmap may be used to visualise the live load effect utilisation history over a long period of time and obtain an overall understanding on the conservativeness of the design live load effect.  

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1Note: An example of spatiotemporal plot in another context (Covid-19 infection rate) can be found in [https://coronavirus.data.gov.uk/details/cases?areaType=ltda&areaName=Cambridge](https://coronavirus.data.gov.uk/details/cases?areaType=ltda&areaName=Cambridge). Accessed 8 October 2021.
Fig. 7.4 Live load effect utilisation heatmaps for the bridge deck – envelope results under the heaviest train measured (Train No. 6): (a) against SLS frequent LM71 load effect; and (b) against characteristic LM71 load effect. Note: (1) no sensor data was available for BM6 to BM8 and for near support locations, and therefore no results are shown for these locations; and (2) each block shows the live load effect utilisation at the centre location of that block (where the FBG sensor is located).
7.4 Visualisation of structural utilisation

Fig. 7.5 SLS live load capacity utilisation heatmap – under the heaviest train measured (Train No. 6). Note: each block shows the live load capacity utilisation at the centre location of that block (where the FBG sensor is located).

Fig. 7.6 Total capacity utilisation heatmap for the bridge deck under the heaviest train measured (Train No. 6): total capacity utilisation (SLS stress) at beam soffit against the tensile stress limit [%]. Note: (1) each block shows the total capacity utilisation (SLS stress) at the centre location of that block (where the FBG sensor is located); and (2) $f_{w}$ is the tensile stress limit at working load.
Fig. 7.7 A demonstration of spatiotemporal live load effect utilisation heatmap for the bridge deck – against characteristic LM71 load effect; envelope results under each of the six train-passage events (Trains No. 1 to No. 6) are used.
7.5 Discussions

7.5.1 Assumptions made in evaluating load effect based on strain data

In order to evaluate the actual load effect (e.g., stress, moment) based on the collected strain monitoring data, assumptions about the material and stiffness properties of the bridge deck (e.g., $E$, $I$) need to be made, for example:

- $\sigma = E\varepsilon$
- $M = EI\Delta \kappa$

It can be seen that the computed load effect is proportional to the assumed material or section stiffness values (e.g., modulus of elasticity, flexural stiffness which is composed of modulus of elasticity and second moment of area). Without measurement data of material and geometric properties, it is important to ensure that the assumptions of $E$ and $I$ are conservative.

The stiffness parameters $E$ and $I$ are subject to variability. The “true” parameter value may fall under a probability distribution function (pdf), such as a normal distribution, which could be derived based on code values, material testing for material properties (e.g., compressive testing on sample cores), and engineering surveying for geometric properties (e.g., laser scanning).

The key question to ask is which end of the pdf is conservative. In other words, is overestimation or underestimation of the particular parameter of interest conservative? For evaluating load effect, it may be conservative to overestimate the load effect, and thus it may be conservative to overestimate the stiffness values when evaluating the load effect. Therefore, the upper tail of the pdf (e.g., 95th percentile for a normal distribution) for $E$ or $I$ may be used for estimating the load effect (e.g., stress, moment).

For the Chebsey bridge, the computed structural utilisation based on monitoring data is still low even if the stiffness values are doubled (which is an extreme and highly unlikely scenario). Therefore, the assumptions of $E$ and $I$ in this case do not affect the general (qualitative or semi-quantitative) conclusion of the study that the live load effect utilisation and live load capacity utilisation of the bridge are low and there is a large degree of conservativeness inherent in the design of this bridge.

7.5.2 Use cases

There are two categories of use cases for monitoring, evaluating and visualising structural utilisation:
1. Inform bridge owners and managers in their bridge operation and maintenance (O&M) decisions

2. Inform bridge engineers in their structural modelling and analysis for bridge assessment (and potentially future design)

As for the first category, more realistic evaluation of structural utilisation based on monitoring data can improve the understanding of the actual “margin of capacity” and therefore inform the decision of “How much additional live load can be placed on the bridge before violating its serviceability, safety or long-term durability criteria?” for traffic management and optimisation.

In addition, understanding which part of the bridge is highly utilised, together with other information such as bridge condition (e.g., damage and deterioration) and structural behaviour (e.g., criticality of local damage to structural integrity), can inform which part of the bridge is critical or vulnerable. This can facilitate more targeted and thus more cost-effective maintenance actions.

As for the second category, the actual structural utilisation based on monitoring data can be used to facilitate the examination of load assumptions (e.g., live load model, load factors) and modelling errors or uncertainties (e.g., the discrepancy between measured load effect and predicted load effect could be due to inaccurate modelling assumptions about load distribution, boundary conditions, etc.). This can facilitate more realistic structural modelling and structural capacity assessment of bridges. In addition, this may be used to identify the sources of conservativeness in the original bridge design and thus inform future design considerations to enable more efficient bridge design.

The results presented in this study have demonstrated the feasibility of monitoring in-service structural utilisation of bridges by enabling more realistic evaluation of different load effects using strain data and comparing them with the corresponding design load effects. The method of evaluating the actual live load effect and comparing it with the design live load effect can be applied to many types of girder bridges (e.g., prestressed concrete girder, steel girder, steel-composite girder) and can serve as a first step to enabling more realistic evaluation of structural utilisation and “margin of capacity”. The method of monitoring and evaluating live load capacity utilisation and total capacity utilisation of prestressed concrete beams can be applied to other prestressed concrete bridges.
7.6 Summary

This chapter has first proposed three definitions of structural utilisation which can inform the "margin of capacity", which is defined in this study as how much additional live load can be safely placed on a bridge without violating the design performance criteria. The chapter has then described and explained the evaluation and visualisation of structural utilisation and discussed the use cases. A summary of the key findings is listed below:

- Three definitions of structural utilisation have been proposed: live load effect utilisation, live load capacity utilisation, and total capacity utilisation. Live load effect utilisation is the ratio between the actual live load effect and the design live load effect; live load capacity utilisation is the ratio between the live load effect and the live load capacity; and total capacity utilisation is the ratio between the total load effect and the total load capacity. These definitions are different in terms of their engineering interpretation, level of complexity and use case.

- The monitoring data collected for the Chebsey bridge can be used to enable more realistic evaluation of two types of load effect, thereby enabling more realistic evaluation of structural utilisation during bridge operation. These are: (i) prestress (based on direct measurement of prestress loss derived from static strain measurements at the top and bottom prestressing tendon of each girder), and (ii) live load effect (derived from dynamic strain measurements under train-passage events). In addition, the load effect of super-imposed dead load predicted in the design could potentially be modified using the partially fixed boundary conditions calibrated based on the monitoring data.

- For visualisation of structural utilisation, three types of plot have been proposed and created: (i) bar plot, (ii) "live" utilisation heatmap, and (iii) spatiotemporal utilisation heatmap. A bar plot can be used to visualise the "margin of capacity" (in particular, how much additional live load may be safely placed on a bridge without violating the design performance criteria) as well as detailed information and breakdown of structural utilisation; a "live" utilisation heatmap is used to visualise the utilisation state across the whole structure at a particular timestamp or during a particular train-passage event; and a spatiotemporal utilisation heatmap is used as a historic summary to visualise historic utilisation and understand past performance.

- For the Chebsey bridge, based on the heaviest freight train measured, the live load effect utilisation and the live load capacity utilisation are low across the whole bridge deck. For example, the maximum live load effect utilisation is approximately 25% against the SLS frequent design live load effect; and the maximum live load capacity
utilisation is approximately 15% at the most critical location (BM5 mid-span bottom fibre) based on the SLS stress criterion. These low utilisation values are mainly due to the conservativeness of the design live load model, the overdesign issue and the conservative assumption of boundary conditions (assumed as simply supported for the Chebsey bridge).

- The visualisation of structural utilisation can enable more intuitive understanding of the structural utilisation results and inform the decision of “how much additional live load may be safely placed on a bridge without violating the design performance criteria” for traffic management. It can also inform, together with other types of information such as structural behaviour and structural damage, which parts of the bridge may be more critical and vulnerable and thus should be paid more attention in bridge maintenance.
Chapter 8

Uncertainty Analysis

8.1 Introduction

This chapter evaluates the levels of uncertainty associated with the various output parameters examined in the studies on in-service structural behaviour and structural utilisation (refer to Chapters 6 and 7) by taking into account the potential sources of uncertainty in sensor data and data processing. These output parameters of interest include neutral axis depth, load effect (e.g., moment) and load distribution characteristics (e.g., normalised curvature). Uncertainty analysis is important for assessing the reliability and robustness of any information extracted from the monitoring data and ensuring that any engineering conclusions reached in bridge performance assessment are of high confidence and thus any associated decisions made are of low risk.

In particular, this study includes three main areas of investigation:

1. Identification and evaluation of potential sources of uncertainty (section 8.2)

2. Propagation of these sources of uncertainty to the output parameters of interest (section 8.3)

3. Sensitivity analysis to identify which sources of uncertainty are significant (section 8.4)

Figure 8.1 presents a workflow of the uncertainty analysis, which also outlines the relationships between the above-mentioned three areas of investigation:

1. First, the potential sources of uncertainty were identified from the bridge monitoring programme, which includes both the sensor system (e.g., types of sensors used, sensor locations, number of sensors) and the data collection method (e.g., data frequency, data collection schedule and duration). The Chebsey bridge monitoring programme was summarised previously in sections 4.3 and 4.4.
2. Second, once the sources of uncertainty were identified, which include both data-related uncertainties and model uncertainties, they were then evaluated. This study focused on the analysis of data-related uncertainties.

3. Third, once the sources of uncertainty were evaluated, they were then propagated to compute the uncertainty levels of the output parameters of interest (in this case, neutral axis, moment and normalised curvature).

4. Finally, a sensitivity analysis was conducted to identify which sources of uncertainty are most significant (i.e., have the largest effect on the uncertainty level of each output parameter of interest). This information can be used to inform and optimise future monitoring programmes for similar bridge assets.

Fig. 8.1 Overview of the uncertainty analysis.

8.2 Sources of uncertainty

8.2.1 Identification of potential sources of uncertainty

Potential sources of uncertainty in sensor data and data processing for a strain-based bridge monitoring programme were identified. Table 8.1 provides a summary of these potential sources of uncertainty, which include both data-related uncertainties and model uncertainties.
### 8.2 Sources of uncertainty

Table 8.1 Summary of potential sources of uncertainty.

<table>
<thead>
<tr>
<th>Category</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Data-related uncertainties</strong></td>
<td></td>
</tr>
<tr>
<td>Random noise of sensor data</td>
<td>• Sensor resolution: i.e., measurement noise</td>
</tr>
<tr>
<td>(i.e., data precision)</td>
<td></td>
</tr>
<tr>
<td>Systematic error of sensor</td>
<td>• Faulty sensors: i.e., erroneous data</td>
</tr>
<tr>
<td>data (i.e., data accuracy)</td>
<td>• Sensor installation error: i.e., what is actually measured ≠ what is intended to be measured, e.g., sensor location error, improper bonding of sensors</td>
</tr>
<tr>
<td></td>
<td>• Data transmission error</td>
</tr>
<tr>
<td></td>
<td>• Sensor calibration error: e.g., inaccurate calibrating parameters</td>
</tr>
<tr>
<td>Effect of data denoising</td>
<td>• Data denoising: reduce data noise but can affect data accuracy</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Effect of data collection</td>
<td>• Data frequency: affect the accuracy of “short-term” response measurement (e.g., dynamic response, transient response)</td>
</tr>
<tr>
<td></td>
<td>• Data collection duration: affect the number of data points available for data post-processing, i.e., “sample size”</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Effect of sensor arrangement</td>
<td>• Sensor locations: affect data magnitude and hence signal-to-noise ratio (SNR)</td>
</tr>
<tr>
<td></td>
<td>• Number of sensors: affect the interpolation errors in data post-processing and the built-in redundancy of the sensor system (i.e., the ability to avoid “single point of failure” should some of the sensors failed)</td>
</tr>
<tr>
<td><strong>Model uncertainties</strong></td>
<td></td>
</tr>
<tr>
<td>Uncertain model parameters</td>
<td>• Uncertainties of the assumed model parameter values: e.g., stiffness parameters ((E, I))</td>
</tr>
<tr>
<td>(i.e., parametric uncertainty)</td>
<td></td>
</tr>
<tr>
<td>Uncertain model structure</td>
<td>• Accuracy of the engineering model(s) used in data post-processing: e.g., model fidelity (“mesh size”), nonlinearity</td>
</tr>
<tr>
<td>(i.e., non-parametric uncertainty)</td>
<td></td>
</tr>
</tbody>
</table>
FBG strain data was used in the in-service structural behaviour and structural utilisation studies (Chapters 6 and 7). Based on Table 8.1, there are four main influencing factors that affect the uncertainty level of any FBG strain data used in data post-processing:

1. random noise of sensor data (i.e., sensor resolution)
2. systematic error of sensor data
3. data frequency
4. data denoising

The following subsection (8.2.2) investigates the four above-mentioned influencing factors for FBG strain data. It should be noted that other sources of uncertainty identified in Table 8.1 are related to data post-processing (i.e., the conversion of strain data to the output parameters of interest) and thus affect uncertainty propagation rather than the data (raw or pre-processed) itself.

8.2.2 Uncertainty level of FBG strain data

Random noise of sensor data

The level of sensor data noise was found from the sensor data of dynamic strains under ambient conditions (i.e., no train-passage event), when no “high frequency” dynamic response was expected. The measured dynamic strains were calculated from the FBG data using Equation 4.2 which assumes zero temperature change in the short time window of interest (e.g., during each train-passage event). Figure 8.2 shows two plots of time-history response from two sensors (BM5 L6 bottom sensor and BM9 L6 bottom sensor, refer to Figure 4.5a for sensor labels and locations) before, during and after a train-passage event. It can be seen that there are high frequency strain variations before and after the train-passage event (i.e., under ambient conditions) when no such dynamic strain is expected. These high frequency strain variations were thus regarded as random measurement noise as a result of the sensor resolution. By comparing Figures 8.2a and 8.2b, it can be seen that the two FBG sensors (BM5 L6 bottom sensor and BM9 L6 bottom sensor) have distinctively different measurement noise.

For this random measurement noise of sensor data, both its magnitude and its distribution are of interest. The magnitude of the sensor data noise was characterised and estimated by calculating the standard deviation (STD) of the sensor data under ambient conditions over a short time window (i.e., no live load and little environmental change). For example, using 2 STDs, the magnitude of sensor data noise was found to be $2.4 \mu \varepsilon$ for BM5 L6 bottom sensor.
Fig. 8.2 Time-history plots of raw strain data before, during and after Train No. 3 (British Rail Class 350): (a) BM5 L6 bottom sensor; and (b) BM9 L6 bottom sensor.
8.2 Sources of uncertainty

and 1.9µε for BM5 L6 bottom sensor. The distribution of the sensor data noise was found by plotting the histogram of the sensor data under the above-mentioned conditions. Figure 8.3 shows the data noise histograms for the two FBG sensors in Figure 8.2. The distinctive “banding pattern” of the FBG sensor data noise was due to the pre-processing algorithm implemented in the fibre-optic analyser used (Lau et al., 2018). A separate unpublished laboratory study by Dr Liam Butler, which used a different type of fibre-optic analyser, found the data noise pattern more “continuous” and closer to a normal distribution with a zero mean.

To characterise and visualise the random noise levels of the sensor data for the Chebsey bridge monitoring programme, Figure 8.4 shows a coloumap of the data noise levels of raw strain data across all FBG sensors in the instrumented girders (BM1 to BM5, BM9). It can be seen that the sensor data noise was clearly not uniform across different sensors. On average, the sensor data noise level was approximately ±2µε (±2 STDs).

Systematic error of sensor data

As previously summarised in Table 8.1, the sources of systematic error of sensor data include faulty sensors, sensor installation error (e.g., sensor location error, improper bonding of sensors), data transmission error and sensor calibration error. There are many existing research studies on detection of faulty sensors, which typically used a data-driven anomaly detection approach (Jeong et al., 2019; Kuok and Yuen, 2020).

In this study, two common types of systematic error for sensor data were investigated for the Chebsey bridge monitoring programme: sensor location error and sensor calibration error. Sensor location error is the difference between the actual sensor location and the intended sensor location. As for sensor calibration error, this is due to inaccurate calibrating parameters (e.g., gauge factors) when converting raw data (e.g., FBG wavelength) to the data of interest (e.g., strain data).

As for sensor location error in the Chebsey bridge monitoring programme, the intended location for the FBG sensors is at the centroid of each instrumented prestressing tendon. In reality, the FBG cable was attached to the underside of the prestressing tendon and surrounded by concrete, and thus the actual location for the FBG sensors (at the centroid of the FBG cable) was different from the intended location. Specifically, each prestressing tendon has a diameter of 13.8 mm and each FBG cable has a diameter of 2 mm (Butler et al., 2016). This means that the difference between the actual location and the intended location for the FBG sensors is −7.9 mm at minimum (the actual location is below the intended location since the sensor cable was attached to the underside of the prestressing tendon), as illustrated in
Fig. 8.3 Sensor data noise histograms (using 600 s of 125 Hz raw strain data under ambient conditions, i.e., no train-passage event): (a) BM5 L6 bottom sensor; and (b) BM9 L6 bottom sensor.
8.2 Sources of uncertainty

Fig. 8.4 Sensor data noise colourmap for raw strain data. Note: white spaces are due to faulty sensors.

Figure 8.5. There were also other influencing factors such as improper sensor attachment and the construction tolerance for prestressing tendon installation.

Table 8.2 shows an analysis of the effect of sensor location error on strain data (refer to Figure 4.6 in Chapter 4 for sensor arrangement at a cross-section). In this example, a relatively conservative value of $-20$ mm sensor location error was assumed ($-10$ mm difference between the prestressing tendon centroid and the FBG cable centroid and $\pm 10$ mm construction tolerance for the prestressing tendon location, which means that the total sensor location error is $-20$ mm to $0$ mm). Based on the dynamic strain data collected for the Chebsey bridge under train-passage events (refer to Table 6.1 for a summary), a hypothetical example of top sensor strain $= -6\mu \varepsilon$ and bottom sensor strain $= 14\mu \varepsilon$ at a cross-section (Refer to Figure 4.6 for the sensor arrangement: top sensor at $z = 600$ mm above beam soffit and bottom sensor at $z = 100$ mm above beam soffit) was used as a benchmark for analysing the effect of different sources of error. These two values were selected due to the ease of calculation and presentation of the results. They also give a theoretical neutral axis level of $450$ mm above beam soffit. Therefore, the same hypothetical example was used throughout this chapter for analysing the effects of other sources of uncertainty.

It can be seen from Table 8.2 that in this selected example, a sensor location error of $-20$ mm (vertical) leads to a strain data error of $0.8\mu \varepsilon$ magnitude. This is $13\%$ of the top strain magnitude ($6\mu \varepsilon$) and $5.7\%$ of the bottom strain magnitude ($14\mu \varepsilon$). It should be noted that
the sensor data error due to sensor location error is an additive error, which means that the data error magnitude is independent of the data magnitude.

Table 8.2 Effect of sensor location error on strain data – an example.

<table>
<thead>
<tr>
<th>Sensor location above beam soffit, z (mm)</th>
<th>Strain (µε)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expected</td>
<td>Error</td>
</tr>
<tr>
<td>600</td>
<td>-20</td>
</tr>
<tr>
<td>100</td>
<td>-20</td>
</tr>
</tbody>
</table>

As for sensor calibration error in the Chebsey bridge monitoring programme, it is affected by the error of each calibrating parameter (refer to Equation 4.1 in Chapter 4 for FBG sensors). For dynamic strain data under live load (assuming little temperature change during each train-passage event), Equation 4.1 was simplified to Equation 4.2 (in Chapter 4), which shows that the dynamic strain data is inversely proportional to the gauge factor, $k_ε$.

Table 8.3 shows an analysis of the effect of FBG sensor calibration error (specifically, $k_ε$ error) on FBG dynamic strain data. In this selected example, a value of 2% $k_ε$ error was assumed. This value was based on a separate unpublished laboratory study conducted by Dr Liam Butler in 2016, which measured the variability and uncertainty level of different
8.2 Sources of uncertainty

calibrating parameters for FBG sensors. It can be seen from Table 8.3 that a 2% error in $k_\varepsilon$ leads to a 2% error in dynamic strain data. This is as expected since the sensor data error due to sensor calibration error is often a multiplicative error, which means that the data error magnitude is proportional to the data magnitude. It should be noted that for FBG static strain data, the effect of each calibration parameter error ($k_\varepsilon, k_T, k_{T_f}, \alpha_{sub}$ – refer to section 4.3 and Equation 4.1) is more complicated and is not strictly multiplicative.

Table 8.3 Effect of sensor calibration error on FBG dynamic strain data – an example.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Assumed value</th>
<th>Value corresponding to</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Minimum $k_\varepsilon$</td>
</tr>
<tr>
<td>Gauge factor, $k_\varepsilon$</td>
<td>0.78</td>
<td>0.74 (95%)</td>
</tr>
<tr>
<td>Strain, $\varepsilon$ ($\mu\varepsilon$)</td>
<td>14.00</td>
<td>14.7 (105%)</td>
</tr>
</tbody>
</table>

**Effect of data frequency on FBG strain data**

Data frequency affects the accuracy of “short-term” response measurement (e.g., dynamic response), this is because a low data frequency may not be able to capture a transient response happening over a short time window. For the Chebsey bridge monitoring programme, one area of interest in structural behaviour and structural utilisation studies was the peak dynamic response (i.e., maximum load effect) under each axle or set of axles during each train-passage event. This subsection investigates the effect of data frequency on the uncertainty level of the captured dynamic strain data, particularly the peak response. This can then be used to inform whether the data frequency is sufficiently high and the minimum data frequency required.

Figure 8.6a shows a time-history plot of strain data under Train No. 6, which is similar to a sine wave response. Figure 8.6b shows a “zoomed in” plot of Figure 8.6a, which includes the strain response under the first few sets of axles. It can be seen based on Figure 8.6b that in order to capture a peak response, there should be enough data points at or around the peak response, which requires sufficiently high data frequency.

To investigate the minimum data frequency required to detect a peak response for a periodical “sine-wave like” response, a theoretical sine wave response was analysed. Figure 8.7 shows one period ($T$) of such a theoretical sine wave response, which has a peak response of 1, a minimum response of 0 (i.e., simulating zero response under no load) and a period of 2.
Fig. 8.6 Time-history response plots under Train No. 6 (BM5 L6 bottom sensor): (a) with the full train-passage event; and (b) “zoomed in” plot – under the first few sets of axles (each dot represents a data point).
The values of these parameters were selected for ease of calculation and illustration. The resulting equation for this theoretical sine wave response is shown below:

\[ y = \frac{1}{2} \sin \left( \left( x - \frac{1}{2} \right) \pi \right) + \frac{1}{2} \quad (8.1) \]

In Figure 8.7, the proportion of the sine wave which is larger than 0.95 (i.e., 95% of the peak response) is highlighted. This proportion was found to be approximately 15% of one period (T). This means that in order to achieve less than 5% error in peak response detection, the data frequency \( f_{\text{data}} \) should be sufficiently high to ensure that there is at least one data point that falls under this highlighted proportion (15%T). In summary, if a “clipping” threshold of 95% is selected, the number of data points at or around the peak response is equal to the number of data points collected during a 0.15T time window, which is equal to 0.15T \( f_{\text{data}} \) (where \( f_{\text{data}} \) is the data frequency).

Fig. 8.7 A theoretical sine wave response (refer to Equation 8.1), with \( y > 0.95 \) (i.e., 95% of the peak response) highlighted.

For the live load response under Train No. 6, \( T \approx 0.7 s, f = 125Hz \), and hence the number of data points at or around the peak response was calculated as approximately 15% \( \times 0.7 \times 125 = 13 \) if the live load response is approximated as a sine wave response. This means that during each period, there are approximately 13 data points which are larger or equal to 95% of the peak response in that period. This also means that to denoise the measured dynamic response using a moving average filter, an N value of less than or equal to 13 would give an error level, for the detected peak response, of less than 5% (upper bound).
and approximately 2% (based on the average value of the 13 data points around the peak response for a theoretical sine wave response).

Based on the analysis above, the upper bound of the percentage error of measured peak response for a theoretical sine wave response was derived, as illustrated in Figure 8.7 and shown in Equation 8.2:

Upper bound of % error of measured peak response = 1 - \[ 1 - \frac{1}{2} \cos \left( \frac{\frac{1}{T_{data}} \times 2\pi}{2T} \right) + \frac{1}{2} \] (8.2)

where \( \frac{1}{T_{data}} \) is the proportion of one period that \( \frac{1}{T_{data}} \) (i.e., the length of time between two neighbouring data points) corresponds to; and \( \frac{1}{2} \cos \left( \frac{\frac{1}{T_{data}} \times 2\pi}{2T} \right) + \frac{1}{2} \) is the response magnitude when the timestamp is at \( \frac{1}{T_{data}} \) away from theoretical peak response timestamp, when the peak response magnitude is 1.

Effect of data denoising on FBG strain data

The noise magnitude of denoised data was characterised and calculated using the same method for that of raw data, as described in the earlier part of this subsection (8.2.2; specifically, “Random noise of sensor data”). Figure 8.8 shows a colourmap of the noise levels of denoised data (using N = 5 moving average) across all FBG sensors in the instrumented girders (BM1 to BM5, BM9). By comparing Figures 8.4 and 8.8, it can be seen that the N = 5 moving average filter reduces sensor data noise by a significant amount (more than 50%). On average, the data noise level for the denoise data was approximately ±6µε (±2 STDs). In addition, data denoising affects the accuracy of dynamic response data (especially peak response magnitude), as previously explained in this subsection (8.2.2).

8.3 Uncertainty propagation

8.3.1 Overview

Once the potential sources of uncertainty were identified and evaluated, these were then propagated to the output parameters of interest. This section investigates the uncertainty propagation process and evaluates the uncertainty levels of three output parameters of interest in the previously deterministic studies on in-service structural behaviour and structural
8.3 Uncertainty propagation

Fig. 8.8 Sensor data noise colourmap for denoised strain data using \( N = 5 \) moving average. Note: white spaces are due to faulty sensors.

utilisation (refer to Chapters 6 and 7). These output parameters are neutral axis, moment (a load effect) and normalised curvature (a load distribution characteristic). The engineering formulae (or “models”) for computing these outputs are summarised in Table 8.4. These have been previously described and explained in Chapter 6 (refer to Equation 6.1 in section 6.2.2 and Equations 6.2 to 6.4 in section 6.2.3).

Table 8.4 Engineering formulae for the three output parameters of interest.

<table>
<thead>
<tr>
<th>Output of interest</th>
<th>Engineering formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>Neutral axis position</td>
<td>( \frac{z_{N, A} - z_{bottom}}{z_{top} - z_{bottom}} = \frac{0 - \Delta \varepsilon_{bottom}}{\Delta \varepsilon_{top} - \Delta \varepsilon_{bottom}} )</td>
</tr>
<tr>
<td>Moment</td>
<td>( M = EI \Delta \kappa ), where ( \Delta \kappa = \frac{\Delta \varepsilon_{top} - \Delta \varepsilon_{bottom}}{z_{top} - z_{bottom}} )</td>
</tr>
<tr>
<td>Normalised curvature</td>
<td>( \kappa_{normalised \ relative \ to \ mid-span, BMi}(x) = \frac{\kappa_{BMi}(x)}{\kappa_{BMi}(x=mid-span)} )</td>
</tr>
</tbody>
</table>

Figure 8.9 provides an overview of the uncertainty propagation process. The left-hand side of Figure 8.9 shows the steps involved in converting raw data to an output parameter of interest. The right-hand side of Figure 8.9 shows the associated sources of uncertainty at each step.
The following three subsections (8.3.2 to 8.3.4) investigate the propagation of data noise, data error and the effect of data collection duration (which affects the number of data points available, i.e., “sample size”), respectively. The uncertainty propagation methodology was then applied to calculate the uncertainty levels of the three output parameters of interest (neutral axis, normalised curvature and moment) in the Chebsey bridge monitoring studies, and the results are summarised and presented (section 8.3.5).

### 8.3.2 Propagation of data noise

Data noise of raw data or denoised data was propagated to the output parameters of interest using either an analytical method (i.e., with an exact closed-form solution) or a Monte Carlo simulation (i.e., apply “the law of large numbers” through repeated random sampling). Monte Carlo simulation was used when an analytical solution was not available.

As for moment, the noise level was computed analytically. This is because moment is proportional to the sum of two independent and normally distributed random variables, $\Delta \varepsilon_{\text{top}}$ and $-\Delta \varepsilon_{\text{bottom}}$ (refer to Table 8.4 for equations). As explained in section 8.2.2, the probability distribution function (pdf) of FBG strain data noise was approximated as a normal distribution; and close to zero correlation was found between the noise levels of different sensors.

The analytical solution for the noise level of moment is summarised below:
8.3 Uncertainty propagation

\[ Z \sim N(\mu_X + \mu_Y, \sigma_X^2 + \sigma_Y^2) \] if \( X \sim N(\mu_X, \sigma_X^2) \), \( Y \sim N(\mu_Y, \sigma_Y^2) \) and \( Z = X + Y \)

Based on this, the output noise level is proportional to the square root of the sum of the data noise squared (i.e., \( \sqrt{\sigma_{\Delta \epsilon_{\text{top}}}^2 + \sigma_{\Delta \epsilon_{\text{bottom}}}^2} \)), where \( \sigma_{\Delta \epsilon_{\text{top}}} \) is the top sensor data noise and \( \sigma_{\Delta \epsilon_{\text{bottom}}} \) is the bottom sensor data noise) for these two parameters.

As for neutral axis and normalised curvature (refer to Table 8.4 for the equations), Monte Carlo simulation was used to calculate the output noise levels because the exact analytical solutions were not available.

Table 8.5 shows the results of data noise propagation (analytical) for moment. Initially, the measurement noise of the raw sensor data was used. Based on Table 8.5, the output noise level is about 14\% of the output magnitude (\( \frac{11}{80} \times 100\% \)), which is relatively significant for computing load effect and structural utilisation. This suggests that the raw sensor data needs denoising to increase the signal-to-noise ratio (SNR) of strain data and thus reduce the noise level of moment results. Based on Table 8.5, the output noise level of moment using denoised data is about 6\% of the output magnitude (\( \frac{5.1}{80} \times 100\% \)).

<table>
<thead>
<tr>
<th></th>
<th>Top strain ($\mu \varepsilon$)</th>
<th>Bottom strain ($\mu \varepsilon$)</th>
<th>Moment (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expected</td>
<td>-6</td>
<td>14</td>
<td>-80.2</td>
</tr>
<tr>
<td>Noise for raw data</td>
<td>2</td>
<td>2</td>
<td>11</td>
</tr>
<tr>
<td>(2 STDs)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Noise for denoised</td>
<td>0.8</td>
<td>0.8</td>
<td>5.1</td>
</tr>
<tr>
<td>data (2 STDs)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Tables 8.6 and 8.7 show the results of data noise propagation (using Monte Carlo simulation) for neutral axis and normalised curvature, respectively. Initially, the measurement noise of the raw sensor data was used. The output noise level of neutral axis is \( \pm 38.4 \) mm (8.5\% of the expected value of 450 mm) and the output noise level of normalised curvature is \( \pm 0.20 \) (20\% of the expected value of 1). These results suggest that the raw data needs denoising to increase the SNR of strain data and thus reduce the noise level of these two output parameters. The output noise levels using denoised data are \( \pm 17.2 \) mm (3.8\% of the expected value of 450 mm) and \( \pm 0.090 \) (9\% of the expected value of 1) for neutral axis and normalised curvature, respectively.
8.3 Uncertainty propagation

Table 8.6 Data noise propagation (using Monte Carlo simulation: 1,000,000 simulations) for neutral axis.

<table>
<thead>
<tr>
<th></th>
<th>Top strain ($\mu\varepsilon$)</th>
<th>Bottom strain ($\mu\varepsilon$)</th>
<th>Neutral axis (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean</td>
<td>-6</td>
<td>14</td>
<td>450.5 *</td>
</tr>
<tr>
<td>Noise for raw data</td>
<td>2</td>
<td>2</td>
<td>38.4</td>
</tr>
<tr>
<td>(2 STDs)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Noise for denoised</td>
<td>0.8</td>
<td>0.8</td>
<td>17.2</td>
</tr>
<tr>
<td>data (2 STDs)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Note: The random number generators in Python are pseudorandom, resulting in a systematic error of +0.5 mm for the neutral axis mean value.

Table 8.7 Data noise propagation (using Monte Carlo simulation: 1,000,000 simulations) for normalised curvature.

<table>
<thead>
<tr>
<th></th>
<th>Beam 1</th>
<th>Beam 2</th>
<th>Normalised curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Top strain ($\mu\varepsilon$)</td>
<td>Bottom strain ($\mu\varepsilon$)</td>
<td>Top strain ($\mu\varepsilon$)</td>
</tr>
<tr>
<td>Mean</td>
<td>-6</td>
<td>14</td>
<td>-6</td>
</tr>
<tr>
<td>Noise for raw data</td>
<td>2</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>(2 STDs)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Noise for denoised</td>
<td>0.8</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>data (2 STDs)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
8.3.3 Propagation of data error

Unlike data noise, data error is inherently systematic, and thus the two types of systematic data error, sensor location error and sensor calibration error (for FBG dynamic strain data), were propagated analytically by directly applying the data error to the strain data and computing the output parameters of interest. As previously described in section 8.2.2, there are two common types of data error, additive error and multiplicative error. Additive error is independent of data magnitude as it is added to the data. Multiplicative error is proportional to the data magnitude and thus, the larger the data magnitude, the larger the error magnitude. In this case, sensor location error is an additive error and sensor calibration error (for FBG dynamic strain data) is a multiplicative error.

Table 8.8 shows the results of the propagation of sensor location error to neutral axis and moment. As explained in section 8.2.2, the total sensor location error could be from $-20$ mm to $0$ mm, which correspond to a strain data error of $+0.8 \mu \varepsilon$ to $0 \mu \varepsilon$. Two extreme cases were considered:

- **Extreme Case 1**: the top and bottom sensors have the same additive error of $+0.8 \mu \varepsilon$.
- **Extreme Case 2**: the top sensor has zero additive error, and the bottom sensor has an additive error of $+0.8 \mu \varepsilon$.

It can be seen from Table 8.8 that Case 1 has a larger effect on neutral axis while Case 2 has a larger effect on moment, although in both cases, the error magnitudes of the two output parameters are small overall.

Table 8.9 shows the results of the propagation of sensor location error to normalised curvature. An extreme case was considered. It can be seen that the error magnitude of normalised curvature is $0.08$ in this case. This is $8\%$ of the expected normalised curvature of 1, which is relatively significant for evaluating load distribution characteristics.

Table 8.10 shows the results of the propagation of sensor calibration error to neutral axis and moment. A multiplicative error of $2\%$ was applied (corresponding to $2\%$ error in gauge factor, $k_\varepsilon$), as explained in section 8.2.2. Two extreme cases were considered:

- **Extreme Case 1**: the top and bottom sensors have the same multiplicative error of $+2\%$.
- **Extreme Case 2**: the top and bottom sensors have equal and opposite multiplicative error ($+2\%$ and $-2\%$).

It can be seen from Table 8.10 that Case 1 has a larger effect on moment while Case 2 has a larger effect on neutral axis (which is different from the effect of additive error), although in both cases, the error magnitudes of the two output parameters are small overall.
Table 8.8 Propagation of the effect of sensor location error to neutral axis and moment.

<table>
<thead>
<tr>
<th></th>
<th>Top strain (µε)</th>
<th>Bottom strain (µε)</th>
<th>Neutral axis (mm)</th>
<th>Moment (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Expected</strong></td>
<td>-6</td>
<td>14</td>
<td>450</td>
<td>-80.2</td>
</tr>
<tr>
<td><strong>Extreme Case 1</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual</td>
<td>-5.2</td>
<td>14.8</td>
<td>470</td>
<td>-80.2</td>
</tr>
<tr>
<td>Error</td>
<td>+0.8</td>
<td>+0.8</td>
<td>+20</td>
<td>0</td>
</tr>
<tr>
<td><strong>Extreme Case 2</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual</td>
<td>-6</td>
<td>14.8</td>
<td>456</td>
<td>-83.4</td>
</tr>
<tr>
<td>Error</td>
<td>0</td>
<td>+0.8</td>
<td>+6</td>
<td>-3.2</td>
</tr>
</tbody>
</table>

Table 8.9 Propagation of the effect of sensor location error to normalised curvature.

<table>
<thead>
<tr>
<th></th>
<th>Beam 1</th>
<th>Beam 2</th>
<th>Normalised curvature (Beam 2/Beam 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top strain (µε)</td>
<td>-6</td>
<td>-6</td>
<td>1</td>
</tr>
<tr>
<td>Bottom strain (µε)</td>
<td>14</td>
<td>14</td>
<td></td>
</tr>
<tr>
<td>Top strain (µε)</td>
<td>-5.2</td>
<td>14</td>
<td>0.92</td>
</tr>
<tr>
<td>Bottom strain (µε)</td>
<td>+0.8</td>
<td>0</td>
<td>-0.08</td>
</tr>
</tbody>
</table>
Table 8.11 shows the results of the propagation of sensor calibration error to normalised curvature. An extreme case was considered. It can be seen that the error magnitude of normalised curvature is -0.04 in this case (which is about 4% of the expected normalised curvature of 1).

Table 8.10 Propagation of the effect of sensor calibration error to neutral axis and moment.

<table>
<thead>
<tr>
<th></th>
<th>Top strain (µε)</th>
<th>Bottom strain (µε)</th>
<th>Neutral axis (mm)</th>
<th>Moment (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expected</td>
<td>-6</td>
<td>14</td>
<td>450</td>
<td>-80.2</td>
</tr>
<tr>
<td>Extreme Case 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual</td>
<td>-6.12</td>
<td>14.3</td>
<td>450</td>
<td>-81.8</td>
</tr>
<tr>
<td>Error</td>
<td>-0.12 (+2%)</td>
<td>+0.28 (+2%)</td>
<td>0</td>
<td>-1.6 (+2%)</td>
</tr>
<tr>
<td>Extreme Case 2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Actual</td>
<td>-6.12</td>
<td>13.7</td>
<td>445.8</td>
<td>-79.5</td>
</tr>
<tr>
<td>Error</td>
<td>-0.12 (+2%)</td>
<td>-0.28 (-2%)</td>
<td>-4.2 (-0.9%)</td>
<td>+0.64 (-0.8%)</td>
</tr>
</tbody>
</table>

Table 8.11 Propagation of the effect of sensor calibration error to normalised curvature.

<table>
<thead>
<tr>
<th></th>
<th>Beam 1</th>
<th>Beam 2</th>
<th>Normalised curvature (Beam 2/Beam 1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top strain</td>
<td>-6</td>
<td>-6</td>
<td>-6</td>
</tr>
<tr>
<td>Bottom strain</td>
<td>14</td>
<td>14</td>
<td>1</td>
</tr>
<tr>
<td>Normalised curvature</td>
<td></td>
<td></td>
<td>1</td>
</tr>
</tbody>
</table>
8.3.4 Propagation of the effect of data collection duration (number of data points)

Section 8.3.2 explains how to calculate the measurement noise level for an output parameter using monitoring data at one timestamp. In practice, data over multiple timestamps are often available. Therefore, the best estimate of the output parameter can be computed by taking the average of multiple data points of the output parameter over multiple timestamps. In general, the larger the number of data points available, the lower the noise level of this best estimate. It should be noted that the number of data points does not affect systematic error of the output.

There are two types of output parameter: structural property and structural response. The former is loading independent, such as neutral axis (under linear elastic behaviour) and normalised curvature (a measure of load distribution). The latter is loading dependent, such as curvature and moment (load effects). The number of data points available (over time) for evaluating a structural property can be much larger than that for evaluating a structural response, since the former does not vary with loading and thus does not vary with time.

The noise level of the best estimate (i.e., the mean value) was obtained by applying the Central Limit Theorem (CLT) and calculating the standard deviation (STD) of the sampling distribution of the sample mean. Specifically, in the context of monitoring:

- Sample: multiple data points of the output parameter
- Sample size \((n)\): number of timestamps of data points available
- Sample mean: mean of the multiple data points of the output parameter, i.e., the best estimate
- Population mean \((\mu)\): true value of the output parameter
- Population STD \((\sigma)\): measurement noise level for one data point of the output parameter

The mean and STD of the sampling distribution of the sample mean, \(\mu_{\hat{X}}\) and \(\sigma_{\hat{X}}\), were computed using Equations 8.3 and 8.4, respectively:

\[
\mu_{\hat{X}} = \mu \tag{8.3}
\]

\[
\sigma_{\hat{X}} = \frac{\sigma}{\sqrt{n}} \tag{8.4}
\]
In the context of monitoring, $\sigma_X$ was regarded as the noise level of the best estimate.

In practice, engineers are often interested in the true value of the output parameter (i.e., the population mean, $\mu$). The information that can be obtained includes the best estimate (i.e., mean of one sample), measurement noise level for one data point of the output parameter (i.e., population STD, $\sigma$; e.g., by applying “the law of large numbers” using Monte Carlo simulation) and the STD of the multiple data points of the output parameter (i.e., STD of the sample mean, $\sigma_X$).

The 95% confidence interval of the true value of the output parameter (i.e., the population mean, $\mu$) was found by applying a two-tailed test in hypothesis testing. This is illustrated in Figure 8.10 and summarised in Equation 8.5:

$$95\% \text{ confidence interval of } \mu = \text{one sample mean } \pm 1.96\sigma_X \quad (8.5)$$

Fig. 8.10 Illustration of the 95% confidence interval of the true value of the output parameter.

### 8.3.5 Application of the methodology: evaluating the uncertainty levels of the three output parameters of interest – neutral axis, normalised curvature and moment envelope

The methodology described in sections 8.3.1 to 8.3.4 was applied to calculate the best estimate and the uncertainty level of each of the three output parameters of interest in the Chebsey bridge structural behaviour and structural utilisation studies (refer to Chapters 6 and 7):
neutral axis, normalised curvature and moment. The first two parameters are structural properties (loading independent) while the third parameter is a type of structural response (loading dependent).

As for structural properties, Table 8.12 provides a step-by-step procedure for computing the best estimate (i.e., “sample mean”) and the uncertainty level for neutral axis. A similar procedure was adopted for normalised curvature. As for structural response, Table 8.13 provides a step-by-step procedure for computing the best estimate and the uncertainty level for moment envelope.

The final results are summarised as follows (using 95% confidence interval for random noise level, and using assumed sensor location error and sensor calibration error for systematic error level):

- Neutral axis (at BM5 L5) = 457 mm ± 15 mm
- Normalised curvature (BM4 L5 relative to BM5 L5) = 1.09 ± 0.12
- Moment envelope (at BM5 L6 under Train No. 3) = 46.2 kNm ± 8.4 kNm
Table 8.12 Step-by-step procedure for computing the best estimate (“sample mean”) and uncertainty level for neutral axis.

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Random noise</strong></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Apply a threshold check on strain data magnitude to ensure sufficient SNR in data (refer to Figure 6.2a): e.g., $</td>
</tr>
<tr>
<td>2</td>
<td>Calculate the number of available data points: 795 data points left for BM5 L5 (near mid-span)</td>
</tr>
<tr>
<td>3</td>
<td>Calculate the mean of these data points: 466.8 mm</td>
</tr>
<tr>
<td>4</td>
<td>Calculate the neutral axis noise level under $</td>
</tr>
<tr>
<td>5</td>
<td>Calculate the noise level of the sample mean by applying $\sigma_{\bar{X}} = \frac{\sigma}{\sqrt{n}}$ (Equation 8.4): $\sigma_{\bar{X}} = \frac{10.2}{\sqrt{795}} = 0.36$ mm</td>
</tr>
<tr>
<td>6</td>
<td>Calculate the 95% confidence interval: $\pm 1.96 \sigma_{\bar{X}} = \pm 0.71$ mm</td>
</tr>
<tr>
<td><strong>Systematic error</strong></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Calculate the systematic error due to sensor location error (-20 mm to 0 mm) under $</td>
</tr>
<tr>
<td>8</td>
<td>Calculate the systematic error due to sensor calibration error ($\pm 2% k_\epsilon$ error) under $</td>
</tr>
<tr>
<td><strong>Total uncertainty</strong></td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Neutral axis = 457 mm ± 15 mm</td>
</tr>
</tbody>
</table>
Table 8.13 Step-by-step procedure for computing the best estimate (“sample mean”) and uncertainty level for moment envelope.

<table>
<thead>
<tr>
<th>Step</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Random noise</strong></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>As an example, select one peak moment (moment envelope): 43.6 kNm for BM5 L6 (near mid-span) under Train No. 3 (refer to Figure 6.3a)</td>
</tr>
<tr>
<td>2</td>
<td>Calculate the moment noise level: [ \sigma = 2.5 \text{ kNm} ] (i.e., 1 STD)</td>
</tr>
<tr>
<td>3</td>
<td>Calculate the 95% confidence interval: [ \pm 1.96\sigma_x = \pm 1.96\sigma = \pm 4.9 \text{ kNm} ]</td>
</tr>
<tr>
<td><strong>Systematic error</strong></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Calculate the systematic error due to sensor location error (-20 mm to 0 mm): -4.8 kNm (worst combination of sensor location error for the two sensors) to 0 kNm</td>
</tr>
<tr>
<td>5</td>
<td>Calculate the systematic error due to sensor calibration error (( \pm 2% \ k_e ) error): ( \pm 0.9 ) kNm moment error (worst combination of sensor calibration error for the two sensors)</td>
</tr>
<tr>
<td>6</td>
<td>Calculate the error due to data denoising (N = 5 moving average): -1% strain data error ( \rightarrow ) -1% moment error ( \rightarrow ) -0.4 kNm moment error</td>
</tr>
<tr>
<td><strong>Total uncertainty</strong></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Moment = 46.2 kNm ( \pm 8.4 ) kNm</td>
</tr>
</tbody>
</table>
8.4 Sensitivity analysis

8.4.1 Overview

One important question in uncertainty analysis is:

Of all potential sources of uncertainty, which ones have significant effects on the output uncertainty level and thus require careful examination and which ones are not important?

Table 8.14 provides an overview of the sensitivity analyses that were conducted and examined.

Table 8.14 Overview of the sensitivity analysis.

<table>
<thead>
<tr>
<th>Uncertainty type</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data noise</td>
<td>Affect output noise</td>
</tr>
<tr>
<td></td>
<td>• Magnitude (e.g., 1µε, 2µε, 5µε)</td>
</tr>
<tr>
<td></td>
<td>• Distribution type (e.g., normal distribution, uniform distribution)</td>
</tr>
<tr>
<td>Data error</td>
<td>Affect output error</td>
</tr>
<tr>
<td></td>
<td>• Additive error: sensor location error (e.g., 10 mm, 20 mm, 30 mm)</td>
</tr>
<tr>
<td></td>
<td>• Multiplicative error: sensor calibration error (e.g., 2%, 5%, 10% error in $k_\varepsilon$)</td>
</tr>
<tr>
<td>Data frequency and data denoising</td>
<td>Affect output noise and output error</td>
</tr>
<tr>
<td></td>
<td>• Data frequency (e.g., 25 Hz, 50 Hz, 125 Hz, 250 Hz)</td>
</tr>
<tr>
<td></td>
<td>• Data denoising: N-point moving average (e.g., N = 2, 5, 10, 20, 50)</td>
</tr>
<tr>
<td>Number of data points</td>
<td>Affect output noise</td>
</tr>
<tr>
<td></td>
<td>Number of data points depends on data frequency and data collection duration</td>
</tr>
<tr>
<td>Sensor arrangement</td>
<td>Affect the accuracy of interpolation using an engineering model</td>
</tr>
<tr>
<td></td>
<td>• Sensor locations (e.g., different vertical locations at a cross-section)</td>
</tr>
<tr>
<td></td>
<td>• Number of sensors (e.g., 2, 3 and 4 sensors at a cross-section)</td>
</tr>
</tbody>
</table>

8.4.2 Summary of uncertainty contribution in three case studies

The contributions of different sources of data-related uncertainty to the uncertainty levels of the three output parameters of interest (refer to section 8.3.5) are summarised in Tables 8.15 to 8.17. Overall, it can be seen from Tables 8.15 to 8.17 that:
1. For a structural property (e.g., neutral axis, normalised curvature), which is loading independent, the output noise level is normally very small since the number of data points available is large. For a structural response (e.g., moment), which is loading dependent, the output noise level is relatively large since the number of data points available is small.

2. Compared with other sources of error, sensor location error and sensor calibration error can have significant effects on the output uncertainty level, and therefore these two sources of error require careful examination both during sensor installation and calibration and during data processing and interpretation.

The following subsections (8.4.3 to 8.4.6) examine the sensitivity of each potential source of uncertainty in more detail, and in particular, the relationship between output uncertainty level and input strain data noise.

Table 8.15 Uncertainty breakdown for neutral axis.

<table>
<thead>
<tr>
<th>Neutral axis [mm]</th>
<th>Breakdown of the output parameter uncertainty</th>
</tr>
</thead>
<tbody>
<tr>
<td>“Sample mean” value = 467 mm</td>
<td>Noise</td>
</tr>
<tr>
<td></td>
<td>Raw data noise</td>
</tr>
<tr>
<td>Uncertainty level *</td>
<td>±38</td>
</tr>
<tr>
<td>% of the “sample mean” value</td>
<td>±8.1%</td>
</tr>
</tbody>
</table>

* Note: For noise level, 95% confidence interval is used; and for error level, extreme cases are used.
8.4 Sensitivity analysis

Table 8.16 Uncertainty breakdown for normalised curvature.

<table>
<thead>
<tr>
<th>Normalised curvature</th>
<th>Breakdown of the output parameter uncertainty</th>
</tr>
</thead>
<tbody>
<tr>
<td>“Sample mean” value = 1.09</td>
<td>Noise</td>
</tr>
<tr>
<td></td>
<td>Raw data noise</td>
</tr>
<tr>
<td>Uncertainty level *</td>
<td>±0.20</td>
</tr>
<tr>
<td>% of the “sample mean” value</td>
<td>±18%</td>
</tr>
</tbody>
</table>

* Note: For noise level, 95% confidence interval is used; and for error level, extreme cases are used.

Table 8.17 Uncertainty breakdown for peak moment (moment envelope).

<table>
<thead>
<tr>
<th>Peak moment [kNm]</th>
<th>Breakdown of the output parameter uncertainty †</th>
</tr>
</thead>
<tbody>
<tr>
<td>“Sample mean” value = 43.6 kNm</td>
<td>Noise</td>
</tr>
<tr>
<td></td>
<td>Raw data noise</td>
</tr>
<tr>
<td>Uncertainty level *</td>
<td>±11.1</td>
</tr>
<tr>
<td>% of the “sample mean” value</td>
<td>±25%</td>
</tr>
</tbody>
</table>

† Note: For noise level, 95% confidence interval is used; and for error level, extreme cases are used.
‡ Note: Only data-related uncertainties are considered here. In practice, model uncertainties (e.g., the assumed values of $E$ and $I$) also need to be considered.
Note: This is not applicable for moment since moment is loading dependent and thus varies with time. Only one data point is available for each moment response.
8.4 Sensitivity analysis

8.4.3 Data noise and data error

The relationship between the noise level of strain data and the noise level of each output parameter of interest was first investigated. For moment, the output noise is proportional to the strain data noise if the strain data noise is uniform across different sensors (refer to section 8.3.2 – if the strain data noise is not uniform across different sensors, the output noise is proportional to the square root of the sum of the data noise squared). This is also illustrated in Figure 8.11a.

For neutral axis and normalised curvature, no analytical solution is available for the relationship between the output noise and the strain data noise. For these two output parameters, Monte Carlo simulations were performed to investigate this relationship. Figures 8.11b and 8.11c present the results for neutral axis and normalised curvature, respectively. In each case, five runs of Monte Carlo simulation were performed to check consistency and reliability. It can be seen that for these two parameters, the output noise level is proportional to the strain data noise level. The results become “chaotic” when strain data noise is above ±6με (±2 STDs) as a result of low SNR.

Another area of interest is whether the assumption of normal distribution for data noise is valid to use. An additional sensitivity analysis on the distribution type of strain data noise was performed. Two distribution types were considered: normal distribution and uniform distribution (an extreme case for the Chebsey bridge monitoring data). Specifically, these are ±2με (±2 STDs) normal distribution and ±1.73με uniform distribution, which are equivalent in terms of STD of strain data noise. It has been found that for each of the three output parameters (moment, neutral axis and normalised curvature), the output mean and output STD values are identical for these two distributions, although the output distribution profiles (i.e., output probability distribution functions) are slightly different.

As for data error, its sensitivity analysis consists of two parts:

1. Effect of sensor location error (additive error) or sensor calibration error (multiplicative error) on strain data error

2. Effect of strain data error on output parameter error

Figure 8.12 and Figure 8.13 provide the results for the first part and the second part of the sensitivity analysis, respectively. The relationship between different input errors and the corresponding output errors can be seen directly from the figures.
Fig. 8.11 Relationship between output noise and strain data noise: (a) moment; (b) neutral axis; and (c) normalised curvature. Note: for (b) and (c), the Monte Carlo simulation results become “chaotic” when strain data noise is above $\pm 6 \mu \varepsilon$ ($\pm$2 STDs) due to low SNR, and thus an upper limit of 200 mm is applied to the vertical axis for presentation purposes.
8.4 Sensitivity analysis

Fig. 8.12 Sensitivity analysis for sources of strain data error: (a) sensor location error; and (b) sensor calibration error (multiplicative error, e.g., 1.0 means no error). Note: compressive strain is +ve.

Fig. 8.13 Data error sensitivity analysis: (a) moment; (b) neutral axis; and (c) normalised curvature.
8.4.4 Data frequency, data denoising and number of data points

As previously explained in section 8.2.2 and illustrated in Figures 8.6 and 8.7, data frequency affects the accuracy of dynamic response measurement. In this study, the effect of data frequency on the measured peak response was investigated. The relationship between the “clipping error” for detected peak response and data frequency, using a theoretical sine wave response as an approximation of live load response (under train-passage events), was previously derived as Equation 8.2 and illustrated in Figure 8.7. This relationship is plotted in Figure 8.14 (to demonstrate the relationship and based on the T values from the measured train-passage events: using T = 0.5 s as an example, with an N = 5 moving average filter applied to the data).

![Figure 8.14: Maximum percentage error of detected peak response as a function of data frequency](image)

The relationship between the output noise level and the number of data points available was previously discussed in section 8.3.4 and shown in Equation 8.4. In summary, the output noise level is inversely proportional to $\sqrt{n}$ where $n$ is the number of data points. This also applies for the relationship between the remaining noise level of moving averaged data and the N value used in moving average. This relationship is illustrated in Figure 8.15 using neutral axis as an example. It can be seen that as the number of data points increases, the output noise level first drops significantly and then quickly starts to have a very small rate of reduction.
8.4 Sensitivity analysis

8.4.5 Discussion on the effect of varying signal-to-noise ratio

As a result of varying load with time (e.g., the variability in live load from the passing trains), the magnitude of dynamic strain varies with time and thus the signal-to-noise ratio (SNR) of dynamic strain data also varies with time. For a structural property of interest (e.g., neutral axis, normalised curvature), which is loading independent, this means that the noise level of this output parameter also varies with time. To reduce the effect of large output noise due to low SNR, a threshold check can be performed on SNR or strain data magnitude (refer to section 6.2 and Table 8.12). On the other hand, such a threshold check reduces the number of data points available, which increases the output noise level (refer to section 8.4.4). Therefore, a balance between SNR and number of data points is needed as they have opposite effects on the output noise level when evaluating a structural property.

Figure 8.16 provides a summary of the sensitivity analysis results for the threshold level in “SNR” threshold check, using neutral axis as an example. Figure 8.16a shows that a higher number of data points and a higher threshold level on strain magnitude give a lower neutral axis noise level. Figure 8.16b shows that as the threshold level of strain data magnitude increases, the neutral axis noise level first decreases and then increases. The lowest point corresponds to the optimal balance between SNR and number of data points.
8.4 Sensitivity analysis

Fig. 8.16 Sensitivity analysis of the threshold level in “signal-to-noise ratio” threshold check: (a) effects of “SNR” threshold and number of data points; and (b) combined effect.
8.4.6 Effect of sensor arrangement

Two of the key questions commonly raised by bridge practitioners for bridge monitoring are:

- Where to put the sensors? (i.e., sensor locations)
- How many sensors? (i.e., number of sensors)

The effect of sensor arrangement (i.e., sensor locations and number of sensors) on the output uncertainty level has been evaluated. In general, more sensors and higher SNR reduce the “interpolation error” when fitting sensor data to an engineering model. In this study, the effects of sensor locations and number of sensors at the cross-section of a prestressed concrete beam (cf. Figure 4.6) on the output parameters of interest (e.g., neutral axis, moment, normalised curvature) were investigated. Similar studies may be performed on the effects of sensor locations and number of sensors along the longitudinal and transverse directions of the bridge deck, although the computation is more complicated due to more complex engineering models involved.

Table 8.18 and Table 8.19 summarise the sensitivity analysis results for sensor locations and number of sensors, respectively, at the cross-section of a prestressed concrete beam. An input data noise of $\pm 2 \mu \varepsilon$ ($\pm 2$ STDs) was applied to the strain data. Table 8.18 shows that sensor locations can have a large effect on output noise level, although the effect is very small in the case of neutral axis. Table 8.19 shows that for these output parameters of interest, the effects of additional sensors on the output noise levels are generally very small.

Table 8.18 Sensitivity of output uncertainty levels to sensor locations at a cross-section (cf. Figure 4.6).

<table>
<thead>
<tr>
<th>Sensor arrangement $(z = \text{distance above beam soffit [mm]})$</th>
<th>Neutral axis (mm)</th>
<th>Moment (kNm)</th>
<th>Normalised curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>$z = {100, 600}$</td>
<td>450</td>
<td>-80.2</td>
<td>1.00</td>
</tr>
<tr>
<td>$z = {0, 600}$</td>
<td>39.8</td>
<td>9.4</td>
<td>0.17</td>
</tr>
<tr>
<td>$z = {0, 900}$</td>
<td>35.4</td>
<td>6.3</td>
<td>0.11</td>
</tr>
</tbody>
</table>

Table 8.18 and Table 8.19 summarise the sensitivity analysis results for sensor locations and number of sensors, respectively, at the cross-section of a prestressed concrete beam. An input data noise of $\pm 2 \mu \varepsilon$ ($\pm 2$ STDs) was applied to the strain data. Table 8.18 shows that sensor locations can have a large effect on output noise level, although the effect is very small in the case of neutral axis. Table 8.19 shows that for these output parameters of interest, the effects of additional sensors on the output noise levels are generally very small.
Table 8.19 Sensitivity of output uncertainty levels to number of sensors at a cross-section (cf. Figure 4.6).

<table>
<thead>
<tr>
<th>Sensor arrangement (z = distance above beam soffit [mm])</th>
<th>Neutral axis (mm)</th>
<th>Moment (kNm)</th>
<th>Normalised curvature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Expected</td>
<td>450</td>
<td>-80.2</td>
<td>1.00</td>
</tr>
<tr>
<td>z = {0, 900}</td>
<td>35.4</td>
<td>6.3</td>
<td>0.11</td>
</tr>
<tr>
<td>z = {0, 600, 900}</td>
<td>29.1</td>
<td>6.2</td>
<td>0.11</td>
</tr>
<tr>
<td>z = {0, 100, 900}</td>
<td>30.1</td>
<td>5.7</td>
<td>0.10</td>
</tr>
<tr>
<td>z = {0, 100, 600, 900}</td>
<td>25.2</td>
<td>5.5</td>
<td>0.10</td>
</tr>
</tbody>
</table>

In addition, it should be noted that the number of sensors also affects the built-in redundancy and thus the robustness of the sensor system. It is important to avoid “single point of failure” scenarios (i.e., if a sensor fails, the whole system fails) for any sensor system. This may be achieved by installing additional sensors at multiple locations.

8.5 Discussion

By systematically identifying and analysing data-related uncertainties of the Chebsey bridge monitoring programme, this study provides a new perspective on: (i) which sources of data-related uncertainty can be most significant in a strain-based bridge monitoring programme; and (ii) how to optimise a strain-based bridge monitoring strategy which includes sensor arrangement (e.g., sensor locations, number of sensors) and data collection (e.g., data frequency, length of time). As an example, Figure 8.15 has demonstrated that the collection of 1000 to 2000 data points (corresponding to 8 to 16 seconds of 125 Hz data) of top and bottom strain sensor measurements under live load may already be sufficient for evaluating neutral axis with a suitable level of precision (e.g., within ±1 mm) and the marginal benefit of additional data points collected over time is small. As another example, Table 8.19 has demonstrated the effect of additional sensors at a cross-section on the output parameter uncertainty level. In this case, for the number of sensors at a cross-section, the effect of a
third or fourth sensor on the output parameter uncertainty level is small in the case of all sensors functioning properly. The additional sensors can be used to increase the built-in redundancy and thus the robustness of the sensor system.

The methodology and results presented in this study can be used to evaluate and demonstrate the uncertainty levels of key output parameters of interest, as a result of various sources of uncertainty in sensor data and data processing. For example, the previous results of evaluated live load effects in Figure 6.4 and Table 6.3 are all deterministic. The evaluated live load effect at L6 location (near mid-span) for BM5 is 43.6 kNm. Based on Table 8.13, it can be seen that the overall uncertainty associated with this value includes an error of $+2.6 \text{kNm}$ and a noise of $\pm 8.4 \text{kNm}$. This additional information of uncertainty level is valuable for assessing the reliability and robustness of the live load effect information extracted from the monitoring data, which helps ensure that any associated conclusions made as a result of this information is robust and of high confidence level in practice.

Overall, based on the findings from this chapter as well as Chapters 5 to 7, the greatest values of the Chebsey bridge monitoring system come from:

1. Early-age and long-term prestress loss evolution can be measured, due to the fact that strain sensors were installed during the manufacturing of the beams and the fact that data were collected over multiple key stages during the bridge’s construction and operation.

2. The longitudinal and transverse bending behaviour of the bridge deck can be measured, due to the fact that the top and bottom sensor measurements of strain were collected for each instrumented beam and the fact that multiple beams were instrumented. This can then be used to evaluate load distribution and better estimate support boundary conditions.

3. Data frequency is sufficiently high to capture the peak response during a train-passage event. It has also enabled the use of a low pass filter (e.g., moving average filter) to reduce data noise to a sufficiently low level and thus obtain sufficiently high signal-to-noise ratio (SNR).

4. Data were collected over multiple train-passage events and enough data points were collected over time to reduce the uncertainty of each output parameter of interest (e.g., live load effect, load distribution, neutral axis) to a sufficiently low level.

As for the design of the monitoring strategy (e.g., sensor arrangement, data collection), the data collection or storage could be more efficient. The data frequency of 125 Hz is appropriate (i.e., not too high or too low) during train-passage events, although it is too high
for other periods or stages. Data collection duration (i.e., the number of data points collected over time) is more than enough for evaluating the output parameters of interest (e.g., prestress loss during operation, live load effect, live load distribution, neutral axis), as demonstrated in the example in section 8.4.4, especially since a large number of data points collected during ambient conditions are not needed. For sensor arrangement, the number of sensors is appropriate to properly capture the bending behaviour of the deck, although instrumenting all nine beams rather than six beams and having more sensors near support locations would be helpful in better capturing the bending behaviour and reducing the uncertainty levels of the evaluated output parameters of interest (e.g., load distribution, support boundary conditions).

8.6 Summary

This chapter has focused on the analysis of data-related uncertainties for the Chebsey bridge monitoring studies (in particular, Chapters 6 and 7). This is important for assessing the reliability and robustness of any information extracted from the monitoring data. Specifically, the study includes three parts: (i) identification and evaluation of potential sources of uncertainty, (ii) uncertainty propagation to evaluate the uncertainty levels of the output parameters of interest, and (iii) sensitivity analysis to identify which sources of uncertainty are significant. Three output parameters of interest were considered: neutral axis, moment and normalised curvature. A summary of key findings is listed below:

- Potential sources of uncertainty in sensor data and data processing for the Chebsey bridge monitoring studies were first identified, which include both data-related uncertainties and model uncertainties. As for data-related uncertainties, these include random data noise (e.g., sensor resolution, effect of data denoising), systematic data error (e.g., faulty sensors, sensor installation error, data transmission error, sensor calibration error, error of measured peak response due to data denoising), influencing factors related to sensor arrangement (e.g., sensor locations, number of sensors) and influencing factors related to data collection (e.g., data frequency, data collection duration).

- Data-related uncertainties for the Chebsey bridge monitoring programme were evaluated. As for random data noise, it was found that overall the raw data noise was approximately $\pm 2 \mu e$ ($\pm 2$ STDs) and the remaining data noise after applying an $N = 5$ moving average filter was approximately $\pm 0.8 \mu e$ ($\pm 2$ STDs). As for systematic data error, it was found that a 20 mm sensor location error (vertical distance) could lead to
a strain data error of 0.8με (±2 STDs) magnitude; and for dynamic FBG strain data, a 5% error in gauge factor ($k_e$) could lead to a 5% error in strain data magnitude.

- Data noise may be propagated to an output parameter of interest using either an analytical solution or a Monte Carlo simulation (a statistical technique). Data error is often systematic and thus may be propagated to an output parameter of interest analytically.

- Data frequency and data denoising can have significant effects on the accuracy of short-term (i.e., dynamic and transient) response measurements, and therefore the data frequency and the data denoising method need to be carefully selected before data collection. As for the use of a moving average filter for data denoising, the selection of an appropriate N value in the moving average filter depends on the data frequency and the short-term response of interest (e.g., peak response).

- As for the effect of number of available data points, $N$, the output noise level is proportional to $1/\sqrt{N}$. This suggests that as the number of data points increases (e.g., as a result of longer data collection duration), the output noise level first drops significantly and then starts to have a very small or even negligible rate of reduction.

- When conducting a threshold check on signal-to-noise ratio (SNR) or data magnitude, a balance between SNR and number of data points is needed, as they have opposite effects on the output noise level.

- Overall, based on the sensitivity analysis results (specifically, the effects of different sources of uncertainty on the three output parameters of interest: neutral axis, bending moment and normalised curvature), of the potential sources of uncertainty, it has been found that sensor location error and sensor calibration error can have significant effects on the output uncertainty levels. Sensor resolution also has a significant effect on the uncertainty level of bending moment. These findings suggest that these sources of uncertainty require careful examination and quality control if a strain-based bridge monitoring programme is to be adopted in the future.
Chapter 9

Conclusions

9.1 Main findings

This PhD research consists of two parts. The first part is based on a series of industry interviews with expert bridge professionals. The objectives are to understand the reasons behind the limited industry uptake of bridge monitoring and model updating in bridge O&M activities and to identify the disconnects between research and practice. The findings from these industry interviews and the literature review form the basis of the second part, which is a study on how monitoring data can be utilised to facilitate more realistic structural capacity and structural utilisation assessment of bridges. In particular, the validity and robustness of some commonly made assumptions about structural behaviour in bridge modelling and analysis have been investigated. An instrumented operational railway bridge was used as the case study. The main findings of this PhD thesis are summarised in the following two subsections (9.1.1 and 9.1.2).

9.1.1 Industry interviews on bridge monitoring and model updating

The important findings from the industry interviews are summarised as follows:

- A key assumption made in the majority of existing bridge monitoring and model updating research for damage detection is that localised damage results in a local reduction in structural stiffness. This is questioned by practitioners as many common types of bridge damage, such as corrosion, may not induce any noticeable change in structural stiffness that existing model updating techniques would identify. The specific types of bridge damage that can or cannot be identified from bridge model updating are not often clearly defined in published research papers.
• Structural model updating is outside the current framework in which bridge practitioners operate to ensure the safety of a bridge structure. In particular, structural modelling in current bridge operation and maintenance (O&M) practice is mostly a one-off exercise rather than routine practice, and it is driven by capacity assessment rather than damage detection. There are also many practical issues related to implementing bridge model updating, such as (i) cost-benefit analysis, (ii) liability issue for bridge owners to keep consultants’ structural analysis models, such as finite element (FE) models, for future use, and (iii) adaptability of these models to future software systems upgrade. These act as barriers to its adoption outside academic research.

9.1.2 Research investigations into more realistic assessment of bridge capacity and utilisation using monitoring data

There has been a mismatch between calculated assessment ratings and the actual demonstrated capacities of bridge assets. Many bridges have been able to remain in service despite large increases in loading above the original design loading. A research study was conducted on how monitoring data can be used to provide confidence in understanding and quantifying the degree of conservativeness inherent in common assumptions used in bridge assessment. More specifically, it involved detailed investigations of measuring structural behaviour and structural utilisation for an operational railway bridge, which has been instrumented with a dense network of both discrete and distributed fibre optic sensors (FOS) since its construction. As part of the study, an uncertainty analysis was also conducted to evaluate the reliability and robustness of any numerical results obtained from the data processing and interpretation.

As for structural behaviour, and in particular, how monitoring data can be used to evaluate certain potentially conservative assumptions in bridge capacity assessment, the important findings are summarised as follows:

• **Prestress Loss**: The remaining level of prestress is a critical parameter in structural capacity assessment of prestressed concrete bridges. Prestress loss can be obtained by measuring the changes of strain in the top and bottom prestressing tendons and evaluating the changes of strain (and stress) at the centroidal level. For the Chebsey bridge, good agreement was found between the unfactored code predictions (based on EC2 and AASHTO) and the FOS measurements of prestress loss, with generally less than 1% discrepancy in terms of percentage prestress loss. This gives confidence in applying the code formula for prestress loss predictions. In addition, both the code predictions and the sensor measurement results indicate that most of the prestress losses occurred during the first three months following the casting of concrete beams.
• **Live Load Distribution**: To investigate the transverse load distribution across different girders under live load, the curvature and bending moment profiles of different girders were normalised relative to one girder (the central girder) in real time to obtain a “normalised” live load response profile and thus enable comparison between these girders. For the Chebsey bridge, the monitoring data showed a near uniform live load distribution across the nine girders under the monitored train-passage events, suggesting very high transverse bending stiffness. This was further validated by the monitoring data of transverse steel cross ties embedded in the bridge deck, which showed negligible strain response during the train-passage events. Overall, the design live load distribution factor (DF) is distinctively higher than the measured DF, which results in a conservative prediction of live load effect (e.g., bending moment under train-passage events).

• **Contribution of Secondary Elements**: The contribution of secondary elements, such as bridge parapets and deck surfacing layers, can be inferred from the degree of partial composite action (i.e., the degree of fixity) between the primary elements (e.g., main girders, main bridge deck) and these secondary elements, based on the monitoring data used to determine the neutral axis positions under live load. For the Chebsey bridge, the monitoring data shows that the edge girders have distinctively higher neutral axis positions than the theoretically calculated values, indicating that the parapets are partially bonded to the bridge deck rather than acting independently as assumed in design.

• **Boundary Conditions**: Boundary conditions at the support bearings were calibrated based on the profiles of normalised curvature (or normalised bending moment) relative to the mid-span position along each girder. This was feasible because the normalised curvature profile along each girder is highly insensitive to the assumptions of material stiffness and axle load magnitude and is primarily sensitive to boundary conditions. For the Chebsey bridge, the monitoring data shows clearly that the boundary conditions are partially fixed rather than simply supported (as assumed in the original design). It has also been found that this is the primary reason behind the discrepancy between the live load effects predicted by the structural model and those effects estimated using the strain monitoring data.

As for structural utilisation, the important findings are summarised as follows:

• **Structural Utilisation and “Margin of Capacity”**: Three definitions of structural utilisation have been proposed to inform the “margin of capacity” (i.e., how much
additional live load can be safely placed on a bridge without violating the design performance criteria): (i) live load effect utilisation (i.e., comparing actual live load effect against design live load effect), (ii) live load capacity utilisation (i.e., comparing live load effect against live load capacity), and (iii) total capacity utilisation (i.e., comparing total load effect against total load capacity). Strain monitoring data has been used to enable more realistic evaluation of different load effects. This can then be used to more realistically evaluate structural utilisation and the “margin of capacity”. For the Chebsey bridge, based on the heaviest train measured, the live load effect utilisation and live load capacity utilisation were low across the whole bridge deck. For example, the maximum measured live load effect utilisation was approximately 25% of the SLS design live load effect at the most critical location (mid-span of central girder). The live load capacity utilisation was approximately 15% of the SLS stress capacity at the most critical location (the soffit of central girder at mid-span). These low utilisations are mainly due to the conservative live load model used, overdesign and conservative assumptions of boundary conditions in the design of the bridge.

- **Visualisation of Structural Utilisation**: Three types of visualisation for in-service structural utilisation have been proposed for three different purposes: (i) bar plot (for visualising the “margin of capacity” and the contributions of different load effects), (ii) “live” utilisation heatmap (for visualising the utilisation state across the whole structure at a particular timestamp or during a particular event, e.g., a train-passage event), and (iii) spatiotemporal utilisation heatmap (for visualising historic utilisation to understand past performance). These plots have been created for the Chebsey bridge, and it has been demonstrated that they can provide clear and intuitive visualisation of structural utilisation and “margin of capacity” for the bridge.

As for uncertainty analysis, the important findings are summarised as follows:

- **Uncertainty Analysis**: The sources of uncertainty for a strain-based bridge monitoring strategy were identified, which include data-related uncertainties and model uncertainties. Data-related uncertainties include random data noise, systematic data error, influencing factors related to sensor arrangement (e.g., sensor locations, number of sensors) and influencing factors related to data collection (e.g., data frequency, length of time). These sources of uncertainty were then propagated to the output parameters of interest (e.g., load effects, structural properties) using either analytical solutions or Monte Carlo simulations (a statistical technique). In addition, a sensitivity analysis was performed to identify which sources of uncertainty are most significant. For the Chebsey bridge, it has been found that sensor location error and sensor calibration...
9.2 Primary contributions

This PhD research, building on previous research on structural model updating and structural behaviour monitoring, provides a systematic methodology of how strain-based bridge monitoring technologies can be used to enable more realistic capacity and utilisation assessment of bridges by quantifying potentially conservative engineering assumptions commonly made in practice.

Specifically, the primary contributions are summarised as follows:

- **Examination of the industry uptake of bridge monitoring and model updating**
  This study, for the first time, provides a comprehensive examination of the reasons behind the limited industry uptake of bridge monitoring technologies and model updating techniques. It also provides a new perspective regarding the future research needed in bridge monitoring and model updating in that it is based on industry practice, industry needs and industry views rather than solely on literature review.

- **Development of a systematic approach to using monitoring data to enable more realistic bridge capacity assessment**
  Building on previous research on bridge model updating and structural behaviour monitoring, a comprehensive and systematic approach to “monitoring-informed bridge assessment” has been developed by demonstrating how monitoring data can be used to evaluate commonly made and potentially conservative assumptions about structural behaviour in bridge modelling and analysis in practice.

- **Development of a methodology of using “normalised” real-time curvature profiles to evaluate load distribution characteristics and calibrate boundary conditions**
  This novel methodology enables automated evaluation of live load distribution characteristics and support boundary conditions in real time under operational conditions. This methodology is better than the conventional methodology of controlled load testing because: (i) no bridge closure is needed; (ii) more data points can be collected over
time to minimise uncertainty and improve reliability of the results; and (iii) the effects of uncertain material stiffness and load magnitude on model updating, particularly boundary conditions calibration, are minimised since normalised curvature profiles are highly insensitive to these two parameters.

- **Development of a systematic methodology for evaluating and visualising structural utilisation of bridges using monitoring data**
  Detailed definitions of structural utilisation have been proposed to inform the “margin of capacity” (i.e., how much additional live load can be safely placed on a bridge without violating the design performance criteria). New investigations have been conducted on: (i) how monitoring data can be used to enable more realistic evaluation of structural utilisation and “margin of capacity”; and (ii) different ways of presenting and visualising the information of structural utilisation and “margin of capacity” based on different use cases.

- **Analysis of data-related uncertainties for strain-based bridge monitoring programme**
  This study provides a new perspective on which sources of data-related uncertainty are most significant in a strain-based bridge monitoring programme as well as how to optimise a strain-based bridge monitoring strategy which includes sensor arrangement (e.g., sensor locations, number of sensors) and data collection (e.g., data frequency, length of time).

These contributions can lead to more realistic and reliable assessment of structural capacity and structural utilisation of bridges using real-time monitoring data.
9.3 Future Work

The development and application of novel or refined methods for using monitoring data to improve bridge assessment, with three areas of investigation on structural behaviour monitoring, structural utilisation monitoring and uncertainty analysis, motivate areas of future research:

- **Improved method for calibration of boundary conditions under operational loading (building on the structural behaviour research)**
  
  As one of the main conservative assumptions in bridge design and assessment, boundary conditions can have a significant effect on load effect prediction and thus load capacity assessment. The method of using normalised curvature envelope profiles adopted in this PhD may be improved by including automatic detection of load patterns (which are relatively sensitive to boundary conditions) and the use of real-time normalised curvature profiles to provide a larger number of data points for calibrating boundary conditions and thus reduce the output uncertainty level. The developed method may then be applied to a large number of train-passage events to test consistency of the calibration results and thus evaluate the reliability of the new method. The starting point could be numerical analyses using different known moving loads as a benchmark study. Once boundary conditions are updated, more accurate and reliable evaluation of live load effects and hence the “margin of capacity” may be obtained.

- **Assessment of immediate and long-term structural risks of increasing live load on bridges (building on the structural utilisation research)**
  
  Further research is needed in order to fully address the question of “How much additional live load may be safely placed on a bridge?” and thus facilitate more efficient utilisation of bridge assets. Specifically, this includes investigating: (i) identification and evaluation of potential immediate and long-term structural risks of increasing live load on bridges; and (ii) how continuous strain monitoring data may be used to better understand and mitigate these risks in near real time while increasing live load on bridges. For example, the immediate risks include violating short-term safety and serviceability criteria, while the long-term risks include reducing remaining service life (e.g., fatigue, durability), inducing potential failure mechanisms and increasing the risk of bridge failure under extreme events (e.g., accidental loads, climate change). How to unlock additional live load capacity while minimising the risks of immediate and long-term bridge failures remains both an industry challenge and an important research problem.
• **Interpretable statistical methods for uncertainty analysis in monitoring data processing and structural performance assessment (building on the uncertainty analysis research)**

Additional research on interpretable statistical methods is needed to build the bridge between engineering interpretation methods that practicing bridge engineers can understand and statistical analysis techniques which can quantify the level of uncertainty (e.g., “level of confidence”) in numerical results and predictions. How to incorporate engineering theory and computational statistics in order to leverage the advantages of both approaches remains a research challenge in the context of bridge monitoring and assessment. The goal is to develop automated uncertainty analysis methods which produce meaningful results that can be interpreted, validated and used by practicing bridge engineers.

• **Identifying, implementing and moving beyond “low hanging fruits” for strain-based structural monitoring of bridges**

More applications and industry pilot projects for strain-based structural monitoring of bridges are needed in order to drive further industry adoption and thus realise the industrial value of bridge monitoring and model updating. Practical lessons learned from these field applications and projects could then be used to further improve the methods developed in this thesis.
References


References


References


References


Mercator Research Institute on Global Commons and Climate Change [MCC] (2021). MCC Carbon Clock.


### Appendix A

#### Literature Survey Summary Table

Table A.1 Surveyed journal papers on model updating of real-world bridges (21 years: 2000-2020)

<table>
<thead>
<tr>
<th>Authors</th>
<th>Year</th>
<th>Country</th>
<th>Bridge</th>
<th>Monitoring data type</th>
<th>Model updating technique</th>
<th>Model updating output (bridge O&amp;M related; actual or intended)</th>
</tr>
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<tbody>
<tr>
<td>Brownjohn &amp; Xia</td>
<td>2000</td>
<td>Singapore</td>
<td>The Safti Link Bridge</td>
<td>✓</td>
<td>✓</td>
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<tr>
<td>Zhang et al.</td>
<td>2001</td>
<td>China</td>
<td>The Kap Shui Mun Bridge, HK</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
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<tr>
<td>Brownjohn et al.</td>
<td>2003</td>
<td>Singapore</td>
<td>Pioneer Bridge</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
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<tr>
<td>Teughels &amp; De Roeck</td>
<td>2004</td>
<td>Switzerland</td>
<td>The Z24 Bridge</td>
<td>✓</td>
<td>✓</td>
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<tr>
<td>Nagayama et al.</td>
<td>2005</td>
<td>Japan</td>
<td>Hakucho Bridge</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>Robert-Nicoud et al.</td>
<td>2005</td>
<td>Switzerland</td>
<td>The Lutrive Highway Bridge</td>
<td>✓ ✓ ✓</td>
<td>✓ ✓ ✓</td>
<td>✓</td>
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<tr>
<td>Zanardo et al.</td>
<td>2006</td>
<td>Australia</td>
<td>MRWA bridge no. 3014</td>
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<th>Country</th>
<th>Bridge</th>
<th>Monitoring data type</th>
<th>Model updating technique</th>
<th>Model updating output (bridge O&amp;M related; actual or intended)</th>
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<tr>
<td>Živanović et al.</td>
<td>2006</td>
<td>Montenegro</td>
<td>A footbridge in Podgorica</td>
<td>✓</td>
<td>✓ ✓ ✓ ✓ ✓</td>
<td>✓</td>
</tr>
<tr>
<td>Daniell &amp; Mac-</td>
<td>2007</td>
<td>U.K.</td>
<td>The Second Severn Crossing</td>
<td>✓</td>
<td>✓ ✓ ✓ ✓ ✓</td>
<td>✓</td>
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<td>Fei et al.</td>
<td>2007</td>
<td>China</td>
<td>Tsing Ma Bridge, HK</td>
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<td>Živanović et al.</td>
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<td>A footbridge in Podgorica</td>
<td>✓</td>
<td>✓ ✓ ✓ ✓ ✓</td>
<td>✓</td>
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<tr>
<td>Caetano, et al.</td>
<td>2008</td>
<td>Portugal and Spain</td>
<td>The International Guadiana Bridge</td>
<td>✓</td>
<td>✓ ✓ ✓ ✓ ✓</td>
<td>✓</td>
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<tr>
<td>Ding &amp; Li</td>
<td>2008</td>
<td>China</td>
<td>Runyang Cable-stayed Bridge</td>
<td>✓</td>
<td>✓ ✓ ✓ ✓ ✓</td>
<td>✓</td>
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<td>Zhao et al.</td>
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<td>Aloisio et al.</td>
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<td>Dong et al.</td>
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Table A.1 – continued from previous page

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References


Appendix B

Industry Interview Questions

1. What are the main structural issues and/or key components of bridges that keep you awake at night? Please rank in order of concern and briefly explain why.
   e.g., bearings, joints, corrosion, fatigue, spalling, scour, flooding, any other key issues?

2. (a) Are you familiar with the idea of structural health monitoring (SHM) for bridges?
   Yes/No

   (b) Do you have any bridge SHM systems installed on any of your bridges / for any of your bridge O&M related activities?
   Yes/No/Don’t know

   Definition: For the purposes of this interview, information from bridge monitoring refers to the collection of any objective data obtained by measuring some physical parameters of the bridge using sensors or other instrumentation, as distinct to subjective data obtained by visual inspection.

   • If yes:
     Please give specific examples, and...
What are the specific reasons for installing them?
What are the benefits of having them in place?
What information is extracted from these SHM systems?
How do you use them to make better decisions?

• If no:
  Why not?

3. What do you think are the key barriers and incentives to using bridge SHM systems to inform bridge O&M?

• Barriers:
• Incentives:

4. (a) Are you familiar with the idea of finite element (FE) model?
  Yes/No

(b) Do you have any FE models of bridges in place for any of your bridges / bridge O&M related activities?
  Yes/No/Don’t know

• If yes:
  Please give specific examples, and . . .
  What are the specific reasons for having them in place?
  What are the benefits of having them in place?
  What information is extracted from these FE models and associated FE analyses?
  How do you use them to make better decisions?
5. What do you think are the key barriers and incentives to using FE models / analyses to inform bridge O&M?

   • Barriers:
   • Incentives:

6. (a) Are you familiar with the idea of finite element model updating, which is sometimes also referred to as structural identification or system identification?
   Yes/No
   Definition: Finite element model updating – matching predicted performance from a finite element model to real performance from sensor measurement data by updating model parameters and/or modelling approaches, in order to create a more accurate “As-Is” model which better reflects the real condition and performance of the engineering structure (in this case, a bridge).

   (b) Has your company used FE model updating for bridges?
   Yes/No/Don’t know
   • If yes:
     Please give specific examples, and . . .
     What are the specific reasons for using FE model updating?
     What are the benefits of using FE model updating?
     What information is extracted from FE model updating?
     How do you use them to make better decisions?
   • If no:
     Why not?
(c) From a bridge engineer and practitioner’s perspective, do you think bridge model updating is a useful exercise and practically solvable problem? Why or why not?

“... The basic principle behind this consists in assuming that localized structural damage results in a local reduction in stiffness. As such, updating stiffness parameters of the FE model in several damage states provides a (non-destructive) means to thoroughly and accurately investigate the condition of the structure.” “... a four-tiered approach to damage assessment: (1) damage detection or identification, (2) damage localization, (3) damage quantification ...” (Simoen, E., De Roeck, G., & Lombaert, G. (2015). Dealing with uncertainty in model updating for damage assessment: A review. Mechanical Systems and Signal Processing, 56–57, 123–149.)

7. What specific future capabilities that are currently lacking (related to bridge condition appraisal and bridge O&M) would you like to see researchers address and develop, which would be useful and add value to your bridge operation and maintenance?