Limitations of Structural Identification: Case Studies

Jeffrey Weidner¹, John Prader¹, Franklin Moon¹, and A. Emin Aktan¹

¹Drexel University
3141 Chestnut St, Philadelphia, PA 19104
(215) 571-4131; jsw24@drexel.edu

Introduction

Structural Identification (St-Id) aims to infer un-measurable but desired responses/attributes of a structure by correlating measured responses with a simulation model. Stated in a slightly different way, St-Id attempts to correlate one or more analytical models with measured data and then uses the correlated models to both estimate current attributes and forecast future performance [1]. St-Id is a valuable tool for aiding in decision-making because when implemented properly, St-Id can estimate key metrics such as live load capacity, seismic vulnerability, etc. that cannot be directly measured or reliably estimated based on heuristics. While this approach is quite promising, there are many potential pitfalls that not only limit its effectiveness, but can also potentially result in highly misleading and unreliable results. This is an unfortunate but undeniable part of the history of St-Id, and has fueled skepticism related to the value of this paradigm.

In contrast to the chosen title, the over-arching goal of this paper is to present a successful case study where St-Id has been applied to a real, operating structure. An honest discussion of some of issues that surfaced during this effort is presented, including how these issues were either directly addressed, mitigated to some extent or circumvented entirely. First a discussion of general limitations and constraints to St-Id is presented, eliminating the need for any repeated discussion in the context of the individual case study. The case study is a “typical” steel stringer bridge, where the focus is on the relationship between response modalities and on practical complications resulting from limited traffic control.

Structural Identification

St-Id is a six-step process, first introduced by Liu and Yao [2]. More recently, the ASCE SEI Committee on Structural Identification of Constructed Systems released a State-of-the-Art report, presenting a detailed discussion of each step and several application case studies [1]. A brief discussion of the process is provided herein.

As shown in Figure 1, the first step of St-Id involves conceptualizing the structure and all concerns and issues that are driving the application. Although this step does not necessarily employ any advanced technologies, it has more influence over the ultimate success of the St-Id than any other as it develops the guiding questions/objectives. In the second step of the process the a priori model is developed and used to predict the bounds of structural responses to aid in the selection of appropriate experimental approaches, sensors, instrumentation plans, etc. This is followed by the actual execution of the experiment (Step 3) - the only objective, quantitative link to the constructed system of interest. While significant advances in sensing and information technologies have been made over the last decade, this step still involves making a series of trade-offs to ensure that the relevant data is acquired with sufficient reliability and without undue cost.

The fourth step involves the processing and interpretation of the experimental data. This step invariably aims to remove data errors and noise (averaging, windowing, filtering), extract key response indices, and plot such indices temporally, spatially, and versus load position, load level, frequency, etc. to facilitate effective interpretation. Once the data has been effectively reduced and interpreted, a simulation model (often from Step 2) is calibrated or updated to minimize its discrepancies with the experiment. The general goal of this process is to reconcile the experiment and model to identify and explain the root causes of the observed data/responses. The final step of the process...
involves using the calibrated model through scenario analysis, parametric studies, or what-if simulations, in order to inform decisions.

**Figure 1 - Structural Identification Paradigm**

### General Limitations of St-Id

St-Id is subject to several general limitations, which are often the source of criticism of the methodology. The most prevalent limitation of St-Id currently is cost, which is currently significant. For signature structures and long span bridges, this cost may seem minimal compared to the annual maintenance budget for the structure. However, even on a typical highway bridge, St-Id requires investment. The operating expenses for conducting a St-Id of most any bridge would include:

- Man-hours for modeling before and after experiment
- Equipment, predominately sensors and cabling
- Man-hours for an on-site team for sensor installation and test execution
- Site access including traffic control, underside access, power supply and loading method
- Travel costs for preliminary site visits and experiment
- Man-hours for data reduction and analysis

There are efforts to reduce these costs wherever possible. This includes new methods like rapid assessment techniques and operational monitoring to inform development of quantitative condition indices [3], technological advances like robust wireless sensing and a trailer-mounted impact device for excitation [4], and automated FE model generation [5]. These advances, while not necessarily geared towards St-Id efforts, may certainly help to reduce some of the one-time costs associated with St-Id.

There is also a significant capital investment required before St-Id can be attempted. The cost of software, data acquisition, sensors, and general installation and testing equipment is large enough that most typical engineering firms would rather outsource to specialty groups or academic institutions as opposed to investing in-house. Even with the capital investment there is still a need for expertise in modeling, instrumentation and data acquisition that is seamlessly coupled with structural engineering heuristics that prevents many practitioners from engaging in these types of efforts.

Finally, St-Id is bridge specific. In the correct framework of testing, inferences about families of structures can be drawn from a single St-Id effort, but this is difficult and potentially misleading if done improperly. This specificity dictates that the cost of St-Id discussed above has to be substantially outweighed by the alternatives (i.e., replacement or retrofit) and/or validated by the potential consequences of inaction (i.e., collapse, substantial impact...
on traffic and commerce). The following case study is presented considering that these criticisms were accounted for and deemed acceptable in the context of the project.

**Case Study: International Bridge Study Structure**

The International Bridge Study encompassed a two year long effort to explore bridge condition assessment with an international perspective. Top researchers and industry partners from all over the world were invited to demonstrate everything from different approaches to bridge condition assessment to new sensor types. The role of Drexel University in this project was to provide a baseline for comparison. The Drexel team led the following experimental investigations as part of the comprehensive baseline St-Id:

- Operational modal analysis study on two spans
- Experimental modal analysis using a sledgehammer and drop-weight for excitation
- Ambient operational monitoring (i.e., strains and displacements)
- Crawl testing at multiple load levels
- Static truck testing at multiple load levels

**General Description of the St-Id Effort**

A single bridge was selected as the test candidate. The structure (Ref), built in 1982, is actually two sister bridges with four spans each. There are a variety of geometric configurations including straight and skewed spans, and a combination of both. The span that was the major focus of the testing was a straight-skew span with a length of 130 ft. on the long edge and 105 ft. on the short edge. The structure consists of eight steel girders with varying flange thicknesses and a consistent web depth of 60 in. The girders are connected laterally through diaphragms, and exterior and first interior girder on each side are connected via diagonal wind bracing connected to the web through a gusset plate 5 in. above the top of the bottom flange. At these gusset plates fatigue cracks have developed. When the cracks propagated into the web, the crack tip was drilled. The girders are supported on pin and rocker bearings resting on concrete pier caps. The overall condition rating for the structure is a 5, mainly due to the condition of the deck. The spans see approximately 93,000 vehicles a day with 4% of that as truck traffic. The span also exhibited noticeably high levels of vibration.

![Figure 2 - IBS Test Structure](image)

The structure was modeled using a typical element-level model consisting of frame elements to represent prismatic components such as beams and diaphragms, and shell elements to represent 2D components such as the deck. The model was rigorously error screened then parameterized to allow for controlled adjustment and variation of critical parameters. Prior to the test, sensitivity studies were run on the selected model parameters (i.e., composite action, bearing stiffnesses, material properties, etc.) and response predictions were developed to guide instrumentation design and provide a measure of confidence during the experiment.
The static testing occurred in the fall of 2010. The instrumentation plan included nearly 100 sensors including mainly strain gages and displacement transducers on a regular grid based on the length of the longest girder. Traffic control consisted of a controlled shutdown of three of the four lanes on the span. The fourth lane was open to traffic since the structure cannot be completely closed. Six trucks in three different configurations were used to load the structure. The three main configurations were three empty trucks side by side, three full trucks side by side, and six full trucks back to back.

The span was successfully loaded to 460 kips with a peak measured deflection of -0.845 in. and a peak bottom flange strain of 183 $\mu$e. While there were numerous interesting and valuable discoveries extracted from this data, the focus of the first part of this paper is on the ability to predict strains and displacements with a model calibrated with modal properties. The span was tested dynamically several times including the same testing window as the static test, but the results used for the model-experiment correlation discussed herein resulted from a test in June 2011. The reason that this test was preferred was that it was the first successful implementation of a custom-built forced excitation drop weight device which imparts a single, broadband impact at a substantial level. Modal parameters were obtained from both experimental and operational modal analysis test procedures, and are discussed in more detail in the following sections.

The a priori model was updated and then calibrated with different modalities of data using traditional manual calibration, parameter identification, and finally Bayesian model updating, a probabilistic approach. The model identified using static data and gradient-based parameter identification was used to explore several scenarios including removal of the wind brace at the fatigue-prone detail. It was shown that removing this detail would have little effect of the operation of the structure and would likely reduce the fatigue problem.

Response Prediction across Modalities

In terms of experiment effort, estimating modal parameters (frequencies and mode shapes) through operational modal analysis tends to be less difficult than collecting displacements and strains through static truck testing, in terms of logistics. For this reason, often an FE model will be calibrated using modal parameters and then used to predict other responses or quantities of interest, like strains used in load ratings. While an FE model is perfectly capable of calculating modal properties as easily as displacements and strains, transitioning between the two can be deceptively difficult.

Recall that the model calibrated using displacement measurements was used for scenario analysis during the St-Id effort described previously. The following discussion considers a model calibrated using the modal properties and then used to predict displacements and strains. The purpose of this investigation was to understand the uncertainty associated with using one modality of response to predict another.

Markov Chain Monte Carlo (MCMC) sampling was used to sample from a given set of parameters included in the model, including composite action, diaphragm stiffness, bearing stiffnesses and barrier/parapet modulus. MCMC randomly generates samples by taking a step from the current sample and then either accepts or rejects the new sample considering the ratio of the likelihood of the new samples given the experimental observations to the likelihood of the current samples. For brevity, the MCMC algorithm is utilized but not presented in detail herein. A thorough discussion of the basic principles of MCMC is included in Gilks, Richardson [6]. The composite action parameter was distributed spatially based on the calibration using the displacements. There were three distinct parameters which could vary independently which made up the composite action. Similarly, the vertical bearing stiffness was also divided into three areas. In general, the northern and southern bearings were each assigned a separate parameter, and Girder #1 was assigned another individual vertical stiffness as a direct result of a crack observed in the pier cap adjacent to the bearing on the southern side.

The parameters were sampled over set ranges with uniform prior probability. The generated samples were accepted at a rate of approximately 35%. The acceptance was based on the likelihood of the model considering its ability to predict the measured modal responses. After the analysis was complete, the accepted models were analyzed under the six truck load stage and response predictions were extracted for comparison to the experiment. The results are
presented in the form of distributions, as opposed to single values. Figure 3 shows the displacement distribution for the midspan of Girder #3 under the six-truck load case, where the maximum response was recorded. The experimental value is shown as the vertical dashed line. The experimental value falls within the distribution, but is located in a low probability area. This indicates that the models, while very capable of representing the modal observations, are not able to predict the displacements accurately.

Figure 3 - Displacement Predictions from MCMC Analysis of Model Informed using Modal Properties

Further investigation into the root cause of this discrepancy showed that certain parameters are much more influential for one type of response than other responses; in this case, the displacements were particularly sensitive.
to the longitudinal bearing stiffness, while the frequencies and mode shapes were not. Figure 4 shows plots of the seven measured frequencies versus the longitudinal bearing stiffness, and versus the midspan displacement of Girder #3. Only one of the seven frequencies shows any correlation, but the midspan displacement is heavily correlated.

**Analytical Mass Scaling of Experimental Mode-shapes for Flexibility Identification**

One of the primary drawbacks of using modal flexibility as a bridge condition indicator is the necessity of developing mass normalized mode shapes. The mass normalization or scaling factor can be extracted from driving point measures where both the input force and resultant acceleration are measured simultaneously. During the testing of the US202/NJ23 test, closure of the structure to all traffic was not feasible, resulting in periods of heavy traffic during testing. All efforts were made to conduct the forced excitation testing during periods where traffic was not present on the structure. However, there were occasions where traffic was on the structure before during or after the impact force was applied. Additional input to the structure other than the impact force resulted in errors in scaling factor estimation, and ultimately the final estimation of modal flexibility.

The possible strategies for calculating accurate modal flexibility from data containing excitation from both impact and traffic include developing a consistent mass matrix from an FE model for mass normalization of the unscaled mode shapes, developing a lumped mass matrix from an FE model, manually developing a lumped mass matrix using tributary area for assigning mass to measured DOF. The first two techniques involve estimating a mass matrix directly from an FE model by applying unit acceleration at one DOF and then calculating the resulting reaction forces at all other DOF in the model.

Once the reaction force at all DOF due to a unit acceleration at a chosen DOF is known, the mass at each DOF can be calculated and populated into the global mass matrix. The FE model likely contains a greater number of DOF than are necessary for comparison with measurements taken during a modal test and any mass matrix derived from an FE model will need to be condensed to match the measured DOF or the measurements will need to be analytically expanded to match the mass matrix from the FE model. For scaling of the mode shapes obtained from the modal testing conducted on the US202/NJ23 Bridge, the FE based lumped mass matrix approach was selected. The feasibility and accuracy of the lumped mass matrix approach was validated using several numerical simulations before implementation on data from the real structure. Once the mass matrix is developed, the un-scaled mode shapes are scaled using the following equation:

$$\Phi = \frac{\phi}{\sqrt{\phi^T M \phi}}$$

\(\phi\) = the unscaled mode shape  
\(\phi^T\) = the transpose of the unscaled mode shape  
\(M\) = the mass matrix of the structure  
\(\Phi\) = the mass normalized mode shape

Once the scaled mode shapes were developed, an interpolation routine was implemented to develop mode shapes at a sufficient resolution to have DOF on the scaled mode shapes correspond spatially with the truck tire positions on the bridge during the static load test. The flexibility matrix was then derived from the high-resolution mass normalized mode shapes using the formulation shown in equation the following equation.

$$[f] = [\Phi] \left(\frac{1}{\omega^2}\right) [\phi]^T$$

\(\Phi\) is the mass normalized modeshape  
\(\Phi^T\) is the transpose of the mass normalized modeshape  
\(\omega\) is the natural frequency  
\(f\) is the flexibility matrix
Comparison with the displacements obtained from the static load test was performed by loading the dynamically derived flexibility matrix with a loading vector representing the weight and the truck tire locations during the load test. A visual comparison between all methods used for estimating the displacement of the structure under known loads is given in Figure 5.

![Displacement Comparison - Gird 3](image)

**Figure 5: Girder 3 Displacements – All Methods – 6 Full Truck Load Case**

Each method is characterized by a maximum error of 8% and an average error of 6%, which shows each method, had reasonable success in predicting the measured displacements along Girder 3. While the figure presents a visual or qualitative comparison of the various methods used for estimating the displacement of the structure under known loading, a quantitative comparison of the errors between the estimation methods is presented in Table 1.

**Table 1: Girder 3 Displacements – All Methods – 6 Full Truck Load Case**

<table>
<thead>
<tr>
<th>Girder - DOF</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>1 vs. 2 (%)</th>
<th>1 vs. 3 (%)</th>
<th>2 vs. 3 (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder 3 - 11</td>
<td>Static Load Test (in)</td>
<td>Modal Flexibility (in)</td>
<td>Ambient Flexibility (in)</td>
<td>1 vs. 2 (%)</td>
<td>1 vs. 3 (%)</td>
<td>2 vs. 3 (%)</td>
</tr>
<tr>
<td>Girder 3 - 11</td>
<td>-0.6157</td>
<td>-0.6047</td>
<td>-0.6543</td>
<td>1.79%</td>
<td>-6.27%</td>
<td>-8.20%</td>
</tr>
<tr>
<td>Girder 3 - 20</td>
<td>-0.8457</td>
<td>-0.7901</td>
<td>-0.8241</td>
<td>6.57%</td>
<td>2.56%</td>
<td>-4.30%</td>
</tr>
<tr>
<td>Girder 3 - 25</td>
<td>-0.5298</td>
<td>-0.4846</td>
<td>-0.6003</td>
<td>8.53%</td>
<td>-13.32%</td>
<td>-23.88%</td>
</tr>
<tr>
<td><strong>Average Error</strong></td>
<td><strong>5.63%</strong></td>
<td><strong>-5.68%</strong></td>
<td><strong>-12.13%</strong></td>
<td></td>
<td></td>
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</table>
Conclusions
The goal of this paper was to first discuss the common limitations of Structural Identification, including mainly cost, logistics, and a limited scope of applicability. Outside of these obvious limitations, there are often more subtle issues that can arise. Two such examples are presented in this paper using the IBS Bridge as a case study.

The first example summarized a study on the IBS structure that used modal properties to inform a Bayesian model updating procedure, and then subsequently predicted displacements. The variance in the displacement results is attributed to discrepancy in the influence of certain parameters on the modal responses as opposed to displacement result predictions. The scenario of informing a model with one type of response and then predicting or calculating others is common in practice and can be misleading.

The second example explains the logistical problem of traffic closures for experimental modal analysis. In the case of the IBS Bridge, one lane of traffic had to remain open during testing. The traffic input polluted the results, making estimation of modal flexibility problematic. The problem was mitigated through estimation of the mass matrix via an FE model. The resulting mode shapes and subsequent flexibility compared well with the experimental results indicating that mass matrix estimation using FE models for experimental modal analysis can mitigate data pollution due to testing during open traffic conditions.

References