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Nondestructive testing for design verification of Boston’s Central Artery underpinning frames and connections

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Prior to the demolition of the Boston Central Artery viaduct in March 2004, a research team implemented a programme of nondestructive testing for design verification of two structural steel highway bents. The tested support bents were used to underpin the original Interstate-93 Central Artery viaduct during construction of the new cut-and-cover tunnel below it. Upon opening of the tunnels, traffic was rerouted from the elevated viaduct to the new tunnel, and the demolition process of the viaduct structure began. Two of the remaining support bents of the underpinning structure were fitted with sensors (strain gages, tiltmeters, slide wire potentiometers, and a 50 kip (222.4 kN) load cell) and loaded by a 50-ton (444.8 kN) crane. The measured structural response was compared to the expected response from finite element structural models, and the structural models were updated using parameter estimation techniques for design verification. Using as-built information, considering original design assumptions, and parameter estimation simulation results, the researchers selected a set of sensor types and locations for the nondestructive field test. Key design parameters of the underpinning finite element model such as connection stiffness values were successfully estimated using structural parameter estimation. As a result, the updated structural response correlated well with the collected nondestructive test data.

Keywords: nondestructive testing; model updating; structural parameter estimation; design verification

Introduction

As part of the Central Artery/Tunnel project in Boston, the existing six lane steel framed Central Artery viaduct was replaced by an 8–10 lane cut-and-cover tunnel. The viaduct remained in service during tunnel construction. To allow for excavation below the viaduct, the existing foundations were underpinned by a series of steel frame bents (Harrington 1998). The new highway tunnel was opened in stages: northbound lanes in March 2003 and southbound lanes in December 2003. With traffic rerouted to the new tunnels, demolition proceeded on the existing viaduct structure. The procedure for demolition was to remove the existing highway superstructure, temporarily exposing the steel underpinning frames before these were also demolished, as shown in Figure 1. Prior to the final demolition, two underpinning frames were the subjects of a series of nondestructive tests (NDTs) to collect measurements for structural design verification and finite element model (FEM) updating. Loading and measuring the performance of the steel bents, Bent 56 (shown in Figure 2) and Bent 57 (shown in Figure 3) provided a unique opportunity for structural in situ testing and response measurements.

The goals of the NDTs were to perform design verification and to perform structural parameter estimation. The process of design verification involved revisiting the original design assumptions and comparing the in situ performance of the structure to the predicted response. Structural parameter estimation involved adjustment of stiffness parameters of the analytical model at the element level to match measured performance. Structural parameter estimation can be used to determine the stiffness-related parameters of a structural member, such as axial rigidity, bending rigidity, and torsional rigidity using applied loads and measured responses. The estimated parameter values are then compared to the design values of the parameters for design verification and model updating.

Structural parameter estimation is the process of reconciling an a priori FEM of the structure with NDT data from the structure. It has great potential for the purpose of damage identification and structural condition assessment of in-service structures, as well as

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design verification and model updating (Santini-Bell et al. 2007). Farrar et al. (2003) and Farrar and Jauregui (1998) summarise the current, state of the art, damage identification methods. In general, structural parameter estimation techniques compare the actual measured response of a structure with the analytical expected response. Both Aktan et al. (1997) and Jang et al. (2002) offer a comprehensive study of the integration of the analytical and the experimental sides of parameter estimation. Multi-response parameter estimation allows the user to combine different types of NDT data collected for a given structure.

Figure 1. Aerial view of the Central Artery underpinning system during deck demolition.

Figure 2. Crane loading for Bent 56.
During this NDT of the underpinning frame, static displacement using a slide wire potentiometer, rotations using Geomechanic Model 900 high gain tiltmeters and strains using ¼-inch strain gages from Vishay measurements were collected (see Figure 4). Multi-response structural parameter estimation combines algorithms for strains (Sanayei et al., 1997) and displacements and rotations (Sanayei and Nelson, 1986).

Design verification involves using nondestructive testing methods on an existing structure to verify assumptions made during the original design phase. Several researchers have investigated issues relating to monitoring and design verification. Xu and Zhu (2000) collected field measurements from the Tsing Ma suspension bridge during Typhoon Victor. The objective of the research was to measure wind behaviour but design verification was not specifically included in the presented research. McElwain and Laman (2000) investigated in-service bridge behaviour in comparison to AASHTO code estimates without performing design verification. Myrvoll et al. (2000) performed a full-scale test for design verification of bridge structures. Tang et al. (2005) used finite element analysis for damage analysis for reinforced concrete arch structures. In Nowak et al. (2000), several bridges were proof-loaded with tanks in order to calculate actual stress response levels. Feng et al. (2004) introduced the idea of a baseline model for structural health monitoring of in-service bridges but does not carry the concept into the initial design of the structure. Shiu et al. (1990) discussed the idea of design verification for a cable-stayed bridge but did not address application to medium-span service bridges.

The research presented is part of a comprehensive research effort, funded by the National Science Foundation, to shift the bridge design paradigm to include instrumentation, testing, design verification and structural modelling (Santini-Bell et al., 2007). Any differences between the assumed design parameters and the estimated parameters can help to reveal the current condition of the structure. Using a discrete mathematical model, such as a FEM, the stiffness parameter estimates reveal not only damage location but also damage severity. Parameter estimation can also determine the current load rating of an in-service bridge accounting for any loss in stiffness during the life of the bridge. It can also be used to predict the remaining life of in-service structures given current loading conditions.

**Nondestructive testing**

Two steel underpinning frames were selected for nondestructive testing. Figure 2 shows the east high pick up frame at Central Artery Bent 56 and Figure 3 shows the west frame at Bent 57 (numbering refers to the original bent designation from the 1950s highway construction, where Bent 1 was the support frame...
furthest to the north at the Charles River). At the time of testing, the I-93 viaduct had been demolished, temporarily leaving the steel support frames in place. This paper will focus on instrumentation, testing and analysis of Bent 57, a moment frame with no cross-bracing. These frames were located at the end of Broad Street in downtown Boston. They were tested after the bridge deck and connecting girders were removed as part of the demolition of the entire elevated viaduct (Figure 1).

Bent 57 consisted of moment-resistant welded connections between steel rolled W-shapes (Figure 5). The columns were W14 × 145 sections oriented in the weak direction (i.e. the flanges are in the same plane as the opening of the frame). The top framing beam was a W36 × 300 section oriented in the strong direction (i.e. the web is in the same plane as the opening of the frame). The columns were connected to the top of the tunnel slurry walls by a bolted moment connection at the base. This connection was assumed to act as fixed for the purpose of structural parameter estimation. Only the upper connections were considered unknown for parameter estimation. The foundation connections were not included in the parameter estimation scenarios because the base condition for Bent 57 was fixed to the ground by a moment connection. The original designers and researchers had great confidence in this connection.

During the underpinning operation, loads from the existing viaduct were jacked into the steel frame structures (Harrington 1998). After the jacking operation, load transfer was accomplished by torch cutting existing steel columns below the new connection points. Underpinning support of the expressway viaduct was in place for approximately 5 years during construction of the cut-and-cover tunnel below. The underpinning frame was fitted with 12 strain gages, three tiltmeters, and a slide wire potentiometer to measure displacement at the mid-span of the frame. Parameter estimation simulations were performed to select strategic sensor locations on the frame (Figure 5) so that the target unknown parameters would be observable during testing (Sanayei and Javdekar 2002). Each sensor location has four strain gages and one tiltmeter so that the bending moment and displacement can be calculated for each location, as shown in Figure 4. Engineering judgement and physical restrictions on the location options were a large part of the measurement selection process (Blanchard 2004).

A field data acquisition system, provided by Geocomp, Inc., was connected to the load cells and all of the sensors on the bent, as shown in Figure 4. Bent 57 was loaded by crane via a system of pulleys and cables. The maximum loads were determined by finite element analysis to ensure that there would be no

Figure 5. Strain gage locations for Bent 57.
out-of-plane displacement. A maximum vertical load of up to 50 kips (222.4 kN) was measured at the contact point on the structure through a load cell (Figure 6a). Physical restrictions of the crane required that a grounded pulley be used to apply lateral loads (Figure 6b). For lateral loading of Bent 57, brackets were welded to the Bent 58 across Broad Street immediately to the south. A maximum horizontal load of up to 25 kips (111.2 kN) was measured at the contact point on the structure through a load cell (Figure 6b). The load was measured using a load cell attached directly to the loading frame. During the NDT of the frame, applied loads, strains, tilts, and displacements were measured at predetermined loading and unloading intervals of 10 kip each (44 kN). Each loading and unloading cycle was repeated three times. Figure 7 shows the load–displacement curve for the top beam of Bent 57 for load case 1. Note that only one displacement measurement was collected during the load test using the slide wire potentiometer.

Both measurements for strain and rotation were reference-independent, so there was no need to measure against a datum. The slide wire potentiometer (SWP) for displacement was not a reference-independent instrument, so it needed to be connected to a datum point. The SWP was secured to the ground and a steel wire was extended and connected to the centre of the W36 × 300 needle beam. A PVC conduit was used to shield the wire from wind-induced vibrations (Figure 3). The strain gages were spot welded onto the frame at specified locations shown in Figure 4. The tiltmeters were clamped onto the structure so that they could be reused. The temperature was not considered in the post-processing of the data given that the actual test for each frame occurred within a 1 h window and the temperature differential was negligible.

Structural model

Figure 8 shows the deformed shape of the structural model for Bent 57 with pinned connections. The model was prepared using GT Strudl®, 29.1 (GT Strudl 2006). Member properties were calculated using as-built shop drawings, noting that the structure was in place for less than 10 years and visual inspection indicated no signs of corrosion or structural damage. Two FEMs were created assuming both pinned and full fixity at column base supports. Table I shows the results from both connections types. Initially, the shear deformation of the beam was ignored, assuming that it was negligible. However, given the depth-to-length ratio of the beam, shear deformation was considered as part of the design verification process. The only unknown parameters considered in the structure were the rotational stiffness values of the two connections between the columns and the beam at the top of the frame.

The structural model of Bent 57 was analysed to produce baseline responses for correlation with the field measurement. The strains, rotations, and
displacements measured from the NDT were charted along with the strains, rotations, and displacements from the FEM for comparison.

Table 1. Summary of simulated results for varying upper connection stiffness values for Bent 57.

<table>
<thead>
<tr>
<th>Connection stiffness</th>
<th>Mid-span deflection, in./cm</th>
<th>Mid-span deflection, cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘Pinned’ 10^4 (11.3 \times 10^6) no moment</td>
<td>0.03822 (0.0976)</td>
<td></td>
</tr>
<tr>
<td>‘Fixed’ 10^7 (11.3 \times 10^9) large moment</td>
<td>0.03692 (0.09194)</td>
<td></td>
</tr>
</tbody>
</table>

Data quality analysis was performed (Blanchard 2004) on the raw data to select the measurement set for parameter estimation and design verification. The data quality analysis was based on the comparison of the measured response with the expected response as well as repeatability patterns within the measured data sets.

Parameter estimation

The Matlab®-based parameter estimation program, PARIS® (PARameter Identification System, Sanayei 1997), developed at Tufts University, estimates the parameters of the elements and connections of a structure’s FEM (Figure 9). The structure can be excited either statically with applied loads, $F$, and...
measuring displacements and rotations, $U$, and static strains or dynamically measuring frequency response function and extracting natural resonance frequencies, $\omega$, and associated mode shapes, $\Phi$, for stiffness and mass parameter estimation. A selected number of measurements gathered sparsely at certain strategically selected degrees of freedom (DOF) were used for parameter estimation.

For this research, only the static stiffness displacement and rotation-based and strain error function was used for parameter estimation. A full explanation of other error functions that are used for multi-response parameter estimation available in PARIS© is given in Santini-Bell et al. (2007). Although this multi-response parameter estimation protocol provides the user with an additional opportunity for data use, the user must also use engineering judgement to ensure that only compatible measurement sets are combined, for example all selected measurement sets must capture the linear-elastic response of the structure. A brief overview of both error functions is presented here.

**Static stiffness-based error function**

The static stiffness based error function, $[E_{st}(p)]$ (Sanayei and Nelson 1986, Sanayei et al. 1992) is developed using the finite element equilibrium, equation (1), for linear elastic structures. Using partitioning and static condensation, the unmeasured data points are removed resulting in an algebraically non-linear error function

$$[F] = [K][U]$$  \hspace{1cm} (1)
\([U_a], [F_a]\) and \([F_b]\) are considered as known measured data. Essentially \([E_{\text{measured}}(p)]\) is residual forces at the measured DOF \((2)\). \([F_{\text{predicted}}]\) is based on the measurements, \([U_a]\), and submatrices of the analytical \([K]\). \([F_{\text{measured}}]\) is the applied set of forces

\[
[E_{\text{measured}}(p)] = [F_{\text{predicted}}] - [F_{\text{measured}}] \tag{2}
\]

The resulting matrix, \([E_{\text{measured}}(p)]\) is NMDOF \(\times NSF\), where NMDOF is the number of DOF measured per load case \((LC)\) and NSF is the number of applied force sets or load cases.

**Static strain error function**

Displacements and rotations are not the only type of measurements that can be collected during a NDT. A NDT can be designed to collect strain data. Strains are typically much smaller in magnitude than displacements and easier to collect in the field. Many researchers believe that strain gauges record more robust measurements and are reference independent. Due to this fact, the static strain error function, \([E_{\text{sstr}}(p)]\), was developed using strain data in the parameter estimation procedure (Sanaye and Saletnik 1996a,b). In equation \((3)\), \([B]\) is a mapping or compatibility matrix that is created using the user-input data regarding the location of the strain measurements along the element in the \(x\)- and \(y\)-direction for a two-dimensional frame element and in the \(x\)-, \(y\)- and \(z\)-direction for a three-dimensional frame element

\[
[e] = [B][U] \tag{3}
\]

The analytical strains, \([e]\) are predicted using applied forces and analytical stiffness matrix, \([K]\). The measured strains, \([e_{\text{measured}}]\), are measured during an NDT. \([E_{\text{sstr}}(p)]\) is based on residual strain measurements

\[
[E_{\text{sstr}}(p)] = [e_{\text{predicted}}] - [e_{\text{measured}}] \tag{4}
\]

**Parameter estimation trials using NDT data**

The assumption of a fixed base connection was confirmed using simulated parameter estimation runs in an attempt to find more appropriate connection stiffness values. For each parameter estimation simulation, a different base condition was used included full, partial, and no fixity. The final base connection stiffness estimated was approximately \(10^{10} \text{ in.-kip/radian}\) \((11.3 \times 10^9 \text{ m-kN/radian})\), which closely approximates a fixed connection, as assumed by the original design. The rotational stiffness of the two upper connections, designed as ‘fixed’ or infinitely stiff, were the main focus of the parameter estimation. By definition, all of the beam-to-column connections of Bent 57, a moment frame, are full penetration welded moment connections. Several iterations of structural analysis were conducted using the FEM of Bent 57 with differing connection stiffness values. Resulting responses with respect to connection stiffnesses are shown in Table 1.

Figures 10 and 11 present the effect of estimated connections stiffness values on the resulting mid-span deflection and connection moments, respectively. These figures indicate that there is a significant change in both deflection and moment when a stiffness of approximately \(10^4 \text{ in.-kip/radian}\) \((11.3 \times 10^6 \text{ m-kN/radian SI units})\) is used and then the rate of change slows down again at \(10^6 \text{ in.-kip/radian}\) \((11.3 \times 10^7 \text{ m-kN/radian})\). Bent 57 was designed as a ‘moment frame’ with the assumption of a stiff connection. The parameter estimates values for Bent 57 using the NDT data are shown in Table 2.

The converged parameter estimates for the connection stiffness values were then used to update the FEM. The structural responses of the three different connection conditions were then compared: pinned, fixed, and the converged condition \((P_{\text{con}})\). \(P_{\text{con}}\) was calculated by PARIS© using the collected measured response. As expected, the deflected shape shown in Figure 12 demonstrates that the converged condition \((2)\) is a very close approximation of the fixed condition \((3)\).

**Beam deflection**

In order to more completely evaluate the NDT data, the effects of shear deflection must be considered in the FEM. Unlike most beams where shear deformation is negligible (Timoshenko and Young 1935), the needle beam for Bent 57 had a depth to length ratio of 3/21, requiring shear deflection consideration. For the deflection of the centre span of the beam due to the vertical centre load, the shear deflection accounts for 22.2% of the total deflection. For the deflection of the centre span of the beam due to the induced shear of horizontal load case 2, the shear deflection is 6.7% of the total deflection.

The FEM did not account for the shear deflection in the frame. Therefore, the NDT data for these deflections was reduced by 22.2% and 6.7%, respectively, for accurate comparison between the NDT data and the FEM. The FEM results were generated by plotting the displacements of points along the beam under different connection stiffness values, resulting in four curves representing the four stiffness conditions (pinned, initial guess, converged, and fixed).
Ideally, the NDT data should fall between the two extreme connection stiffness conditions – pinned and fixed. As a verification of the NDT data points, the FEM curves were re-calculated including shear deflection and were compared to the unmodified NDT data. The simulated mid-span deflection values are shown in Table 2 and compared with the raw NDT deflection, 0.03842 inches (0.0976 m). The large shear area of the W36 × 300 allows the NDT data to be evaluated with respect to the shear displacement with confidence, and the plots with the post-processed

<table>
<thead>
<tr>
<th>Connection</th>
<th>Initial stiffness value, in.-kip/radian (m-kN/radian)</th>
<th>Final parameter estimate, in.-kip/radian (m-kN/radian)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Beam-to-column</td>
<td>$10^5 (11.3 \times 10^3)$</td>
<td>$9.58 \times 10^7 (10.8 \times 10^6)$</td>
</tr>
<tr>
<td>Column-to-footing</td>
<td>$10^{10} (11.3 \times 10^9)$</td>
<td>$10^{10} (11.3 \times 10^9)$</td>
</tr>
</tbody>
</table>

Figure 10. Variation of mid-span deflections due to differing connection stiffness values.

Figure 11. Variation of induced moment at connection due to differing connection.
NDT data and FEM-generated curves can be used with confidence as well.

**Connection moments**

Strain data from the NDT was transformed into moment values at the locations where the strains were measured. The calculations for this transformation excludes the strain due to member elongation (from axial force) to determine the induced moment at that point. The calculated moments were compared with the FEM moment curves to determine how close the FEM approximated the NDT data. FEM moment diagrams were generated by determining the moment at several points along the member for the four connection stiffness cases: pinned, initial, converged and fixed. Figure 13 shows the moment diagram of the beam due to load case 1; the envelope between the extreme conditions (pinned and fixed) is very small. Thus, any measurement error in the NDT strain data would have a significant effect on the moment value calculated for the NDT data. This NDT data point is a relatively good match for the model that had been updated with the converged stiffness case.

**Parameter estimation results**

Overall, the updated model indicates a general trend that the converged connection stiffness was a very close approximation to the fixed condition in all load cases and members. Also, the initial condition was a very close approximation to the pinned condition, illustrating that the initial guess at the stiffness of the beam-column connections was not close to the value it finally converged upon, and that the value the parameter estimation runs returned was a very close approximation of the design assumptions made by the engineers that designed the bent.

The FEM stiffness–curvature curves were generated by calculating deflection (or moment) at several locations along a structural member with four beam-column connection stiffness values (pinned, initial, converged, and fixed). The NDT data was plotted on the same axes as the FEM curves for a direct visual comparison to the FEM curves. In the beam portion of the frame due to load case 1, the envelope between the
extreme connection stiffness conditions (pinned and fixed conditions) was very small, but the NDT data still fit very closely to the FEM stiffness-curvature curves.

In general, the trends shown in Figures 12 and 13 are consistent with the design assumptions. The NDT data typically reflects the fixed condition curves. The FEM updated with the parameter estimation results was a vast improvement over the initial guess at the unknown parameter values, which was the goal of this analysis. The parameter estimation value returned for the stiffness of the upper connections closely mimics the behaviour of the structure as if these connections had infinite stiffness (fixity). The real stiffness of the upper connections is likely very close to the converged value of $9.58 \times 10^7$ in.-kips/radian $(10.8 \times 10^6$ m-kN/ radian) because when the FEM model was updated with the converged parameter values, the FEM curve moved away from the pinned condition curve and very close to the fixed condition curve. An improvement of this magnitude between initial and converged FEM curves shows the parameter estimates reflect the field conditions.

**Conclusions**

NDT data obtained from the Central Artery was used for parameter estimation and model updating. Using PARameter Identification System (PARIS®), the static stiffness-based error function with the NDT data from the underpinning bents of Boston’s Central Artery proved successful. The rotational stiffness values of the connections between columns and beam were successfully estimated. The success of the parameter estimation is verified in that a similar value was converged upon from several different initial parameter values (showing that starting at any point, the minimum residual between NDT data response and simulated response could be found). Also, the estimated parameter values verified the connection stiffness simple design assumptions made by structural engineers.

Bent 57 was considered to be a fixed connection (high rotational stiffness), so the initial parameter ‘best-guess’ was a low rotational stiffness (to prove the high value could still be converged upon). The value that was converged upon was input into a FEM that showed a structural response close to that of a fixed connection. The FEMs of Bent 57 were then compared to the NDT strain data that was considered to be accurate data. Generally, the NDT data agreed with the updated FEM model. There were a few cases that the inherent measurement error in the NDT data exceeded the difference between the predicted FEM results and the collected NDT data.

This research illustrates that, via parameter estimation and model updating, the final parameter estimates were able to more closely approximate the design assumptions made by design engineers, as reflection through NDT data. This paper presents a small-scale proof-of-concept example of structural parameter estimation for design verification. The authors plan to take the lessons learned through this example, including sensors placement, data quality, and realistic expectations from parameter estimation to design verification for a full bridge structures using NDT data and structural parameter estimation.

The lessons learned from this research, NDT, and parameter estimation and model updating will be used for future projects such as a load test that took place in April 2008 at the Rollins Road Bridge in Rollinsford, NH. This test was conducted by the University of New Hampshire in conjunction with the New Hampshire Department of Transportation (NHDOT). The Rollins Road Bridge Project will be used as a benchmark for the structural health monitoring and asset management programme at the NHDOT.

**Acknowledgements**

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