Using Structural Monitoring in the Evaluation of the A. Murray MacKay Bridge

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ABSTRACT

The A. Murray MacKay Bridge is a landmark bridge structure in Halifax Harbour and a vital transportation link between Halifax and Dartmouth for both passenger vehicles and heavy traffic. The bridge, which includes a central suspension bridge structure and two approach structures, was opened to traffic in 1970.

Halifax Harbour Bridges conducts a regular evaluation of the condition and safety of its bridge structures. Buckland & Taylor Ltd was contracted to conduct one such load evaluation on the main suspension elements of the MacKay Bridge in accordance with the provisions of the most recent version of the Canadian Highway Bridge Design Code and current traffic demands.

This evaluation included the development of a three-dimensional finite element model of the bridge and its response to truck live loads. To supplement this theoretical work, a joint team of Remote Access Technology Limited and the Centre for Innovation in Infrastructure at Dalhousie University has installed a structural monitoring system on key elements of the stiffening trusses of the bridge. The structural monitoring data is being used to calibrate the theoretical model and develop better peak stress estimates based on in-situ response to random daily traffic loads.

The paper demonstrates how advances in engineering tools allow us to collect more actual data on in-situ conditions and behaviour. Rigorous analysis and structural monitoring can be effectively integrated into the bridge management process. Better in-situ condition assessments will lead to decisions that can be made based on knowledge and not on uncertainty.

INTRODUCTION

The A. Murray MacKay Bridge (MacKay Bridge) is a landmark bridge structure in Halifax Harbour and a vital transportation link between Halifax and Dartmouth for both passenger vehicles and heavy traffic. The bridge was designed and constructed between 1966 and 1969 and opened to traffic in 1970. The bridge is a total length of approximately 1200 m with suspended spans totalling approximately 740 m. The bridge supports 4 lanes of traffic with average daily vehicle crossings exceeding 50,000. The operation and maintenance of the bridge is conducted through Halifax Harbour Bridges (HHB), a provincial government commission. Its mission is to provide safe, efficient and reliable passage at an appropriate cost.

To fulfill its mission, HHB must base maintenance and management decisions on sound engineering practice coupled with accurate specific knowledge of the MacKay Bridge condition and performance. As part of its mandate, HHB conducts a regular evaluation of the condition and safety of its bridge structures. Buckland & Taylor Ltd (B&T) was contracted to conduct one such load evaluation on the main suspension elements of the MacKay Bridge in accordance with the provisions of the most recent version of the Canadian Highway Bridge Design Code and current traffic demands.

In addition to visual and physical assessments, this evaluation included the development of a three-dimensional finite element model of the bridge and its response to truck live loads. To supplement this theoretical work, a joint team of Remote Access Technology Limited and the Centre for Innovation in Infrastructure at Dalhousie University (RAT/DAL) has installed a structural monitoring system on key elements of the stiffening trusses of the bridge.

This paper discusses the background and rationale for choosing to implement a structural monitoring system to supplement load evaluation work. The use of the model analysis to identify the most appropriate monitoring plan is presented. The installation and operation of the monitoring system is reviewed. Finally, the use of the monitoring data from controlled testing to calibrate the theoretical models is discussed. Data from on-going regular monitoring is also presented.

The objective of the paper is to demonstrate how advances in engineering tools allow us to collect more actual data on in-situ conditions and behaviour. Adjusting to new realities requires that new techniques and ideas be appropriately and effectively integrated into bridge maintenance and decision making. Management decisions can therefore be made based on knowledge and not on uncertainty.

MACKAY BRIDGE

The A. Murray MacKay Bridge, shown in Figure 1, spans Halifax Harbour between Halifax on the west side and Dartmouth on the east side. It carries four lanes of traffic, with no pedestrian sidewalks, although maintenance access walkways exist below deck on the approach spans and have recently been added to the suspended spans on the north side of the deck at traffic level and on the south side of the deck at truss bottom chord level.

The bridge consists of three main structural forms: the Halifax Approach span (shown in Figure 2), the Suspension Bridge (shown in Figure 1) and the Dartmouth Approach Span (similar to the Halifax Approach span). The stiffening trusses of the deck of the Suspension Bridge are the main focus of the work reported in this paper.

Suspension Bridge Details

The Suspension Bridge shown schematically in Figure 3 is approximately 740 m long and comprises the following components: Halifax Side Span (HSS) of approximately 156.6 m; Centre Span (CS) of approximately 426.8 m; and Dartmouth Side Span (DSS) of approximately 156.6.

Each tower is constructed of steel, is approximately 88.4 m high above the top of the foundation and consists of two legs. Each leg is formed from three hollow cells arranged in a cruciform shape. The legs are joined to each other by horizontal struts and diagonal bracing (horizontal struts exist only at the tower tops and just below deck level).

Each cable consists of 61 galvanized steel wire strands approximately 40 mm in diameter arranged hexagonally. Cedar fillers are used on the flat sides of the hexagon to create a round profile. This is then wrapped in galvanized wire and painted to protect the cable strands from the elements. The Halifax and Dartmouth ends of the cables are anchored in the massive cable anchorage concrete chamber under each respective Approach.

Vertical wire rope hangers spaced at 9.65 m (every other truss panel point) suspend the deck from the cables. These hangers originate and terminate at deck level and pass over cable bands clamped to the main cables.

The deck system consists of two under-deck longitudinal stiffening trusses shown in Figure 4 that support transverse stiffening trusses at every longitudinal truss bottom chord panel point. The top chords of the transverse stiffening trusses are the deck floor beams. The floor beams, in turn, support an orthotropic steel deck (9.5 mm thick deck plate supported by U-shaped longitudinal stiffeners spaced at 600 mm transverse spacing). The steel deck acts compositely with, and forms part of, the top chords of the longitudinal and transverse trusses. Lateral bracing is provided in the plane of the bottom chords.

EVALUATION PROCEDURE BY B&T

For the suspension bridge, the evaluation procedure consisted of:

i. Determining appropriate bridge loadings (both those of the original design and those currently mandated by the Canadian Highway Bridge Design Code, (CHBDC), CAN-CSA-S6-06);

ii. Developing a computer model to provide member Demands (D's) for these loads;

iii. Calibrating the output of the computer model subjected to original design loadings;

iv. Developing current Demands based upon CHBDC requirements;

v. Calculating member Capacities (C's) based upon CHBDC formulations;

vi. Calculating, for each member for each loading type, the resulting Demand/Capacity (D/C) ratio; and

vii. Calculating, for each member for each loading type, the resulting Live Load Capacity Factor (LLCF).

A D/C ratio greater than 1.0 indicates that the demand produced by the specified combination of loads exceeds the capacity of the member based upon the target level of safety specified by CHBDC Section 14. A D/C greater than 1.0 does not necessarily imply member failure – it does indicate that a member provides a level of safety, under the specified load, that is less than the target level of safety defined in CHBDC Section 14.

Using the prescribed loads, calculated member capacities and the corresponding load and resistance factors, a Live Load Capacity Factor (LLCF) was calculated for each member. The LLCF is a ratio of the maximum acceptable live load to the live load being considered (the evaluation load). The LLCF indicates the portion of the evaluation live load that can be carried by the structure's members at the required level of safety. An LLCF of 1.0 or greater indicates that the member capacity is adequate for the demand produced by the evaluation load.

An LLCF of less than 1.0 indicates that the member capacity is inadequate for the demands imposed by the evaluation loads and indicates what portion of the evaluation load could be safely carried. An LLCF of less than 1.0 does not necessarily imply member failure – it does indicate that a member provides a level of safety under the evaluation load that is less than the required level of safety defined in CHBDC Section 14.

A key element in the load evaluation is the development of a numerical model of the Suspension Bridge. B&T used an existing three-dimensional computer model of the suspended spans of the MacKay Bridge shown in Figure 5. The model was created for use with the B&T in-house structural analysis program "CAMIL", and post-processing of the CAMIL output results was carried out using the B&T in-house program "ERC."

All members in the towers, cable bents, stiffening trusses, floor trusses, and laterals were explicitly modeled using beam members, while the main cables and hangers were represented by nonlinear cable elements. The axial and lateral bending stiffnesses of the ribbed plate in the orthotropic deck was incorporated using weightless beam members connected along the bridge centerline to the floor truss top chords. The orthotropic deck and top chord members were therefore modelled by three spines (three lines of beam members), with the outer spines containing the properties of the stiffening truss chord members and the central spine containing the properties of the ribbed plate. Top chords of the transverse floor trusses were modeled with high stiffness to transfer axial loads to the central spine. A portion of the deck model showing the stiffening trusses is illustrated in Figure 6.

For effective theoretical load evaluation, it is important that the numerical model be as accurate as possible. This is achieved by model refinement and validation through calibration. Initially, the global numerical model was assessed by analyzing the bridge for the loading assumed for the original design and comparing the resulting forces and deflections with those given on the original design drawings. B&T carried out this calibration by choosing several original design load cases to apply to the global model, and comparing the resulting member force effects to those shown on the drawings. The result of the calibration was confirmation that the analysis model was providing member demand results consistent with those determined by the bridge's original designers.

For the deck in particular, vehicle live load represents the critical loading conditions. In accordance with Clause 14.9 of CHBDC, live load corresponding to Evaluation Level 1 was used in this study. For this Evaluation Level, CHBDC Clause 14.9.1.2 specifies the live load to be the CL1-W truck or the associated lane load. The CL1-625 was used as the Evaluation Truck and the uniform load part of the CL1-W Lane Load was taken as 9 kN/m, which is the required loading for Class A highways.

Based on these modelling parameters, the theoretical D/C and LLCF ratios were determined for elements of the deck and trusses, as well as towers, cables, hangers and bearings. In many circumstances, the results of this level of evaluation would be used for planning upgrade and maintenance to a structure. However, as maintenance to even just the deck system for a bridge the size of the MacKay Bridge can cost in the tens of millions of dollars, HHB elected to conduct additional calibration work to assist in more effective decision making.

CALIBRATION AGAINST ACTUAL LIVE LOAD BEHAVIOUR

Although the modelling and analysis activity was quite rigorous, there are still elements of uncertainty in the load evaluation. The initial calibration by B&T was conducted using predicted member forces and global deflections from the as built drawings. It was decided that additional calibration work should be conducted using actual local strain measurements under controlled loading conditions. In addition, it was decided to evaluate the maximum strains developed under ambient traffic loads against those predicted by the multiple vehicle and lane load models used in the theoretical evaluation.

To assist in this aspect of the calibration procedure the RAT/DAL team was engaged to conduct real-time structural monitoring of the deck system. The load evaluation model developed by B&T was very important in planning the structural monitoring plan. The members with the most critical D/C and LLFC ratios were identified and selected for monitoring. For the MacKay Bridge deck system, these elements were the stiffening truss diagonals (seem in Figure 4) and the lateral bracing elements. As only a select number of components can be economically and practically monitored, the results of the modelling indicated that the instrumented section should be in the main suspended span close to the Halifax Tower as shown in Figure 3.

Instrumentation

In the final structural monitoring plan, 10 stiffening truss diagonals (see Figure 7) and 2 lateral bracing elements (close to Diagonal 1 and Diagonal 8) were selected for monitoring. Linear pattern weldable strain gauges (Vishay Micro-Measurements General Purpose Strain Gauges type LWK-06-W250B-350) were installed on each element. A second redundant gauge was installed at each location to ensure data integrity. Each gauge was connected to a Campbell Scientific CR-5000 datalogger using 3 conductor, jacketed and shielded cable. The three conductor arrangement reduces noise and thermal influence on the gauge readings due to the long lengths of cable required to connect the gauges to the datalogger. The gauges are continuously interrogated every second. Data files are automatically transferred via wireless modem to a server at Dalhousie University every hour.

Live Load Calibration Testing

Two identical live load calibration tests were conducted in December 2009 and March 2010. During the calibration test, the bridge is closed to all traffic. A test truck, shown in Figure 8, was then positioned at ten predetermined positions and strain data collected for each position. The locations of these positions represents the distance of the front tire from the centerline of the Halifax Tower are given in Table 1. This controlled static testing was then repeated in each of the three remaining traffic lanes of the bridge.

Once the static loading testing was completed, the test truck was driven in each of the four traffic lanes at two different speeds, 10 km/h and 40 km/hr, to simulate slow and fast traffic, respectively.

Calibration Test Results

The data from these calibration tests was used in several comparisons. One important comparison was the stability and repeatability of the data collected from the monitoring system. Just as the theoretical model must be reliable so too must the in-situ strain data. The strain response recorded at the gauge on diagonal 1 for the static testing results from December and March is plotted in Figure 9. The solid lines represent the December test results for the truck in each traffic lane and the dashed lines represent the March test results. These gauge results are typical of all other gauges. It is worth noting that the average air temperature during the December tests was -20℃ while the air temperature for the March tests was approximately 2℃. The strain response shows excellent agreement in all four traffic lanes demonstrating the stability of the instrumentation.

A typical strain response (Diagonal 9) for the test vehicle moving at slow speed in Lane 1 is shown in Figure 10. This data helps to establish a complete influence line for the design truck. The results of the moving vehicle data and the static test data for this gauge are compared in Figure 11. There is again very good agreement confirming the reliability of the monitoring data.

Comparison with Theoretical Model

The data from the static calibration tests was compared with the predicted results for the same loading from the B&T numerical model for Diagonal 6 in Figure 12. Similar to Figure 9, the response for the vehicle in each of the four traffic lanes respectively is shown. Both the measured and the theoretical results produce the same shape of the strain response as the vehicle is moved through the various positions on the bridge. They also show the reduction in member strain as the vehicle is placed in different traffic lanes transversely farther away from the instrumented stiffening truss. Using the peak strain readings for each traffic lane, the lane reduction ratio (Lane X/Lane 1) was computed. These values are shown for diagonal 6 in Table 2.

The correlation in lane reduction for transverse load position between measured and predicted is also very good. This indicates that the model is predicting the overall response of the bridge very well.

As mentioned, each diagonal was instrumented with two gauges to create redundancy in the measurements. In many cases, the agreement between the two gauges was very good as shown in Figure 13 for the two gauges on Diagonal 6. The difference in peak strain for the truck in Lane 1 (the highest strains) between the two gauges is 2 με or approximately 1% of the maximum. In other cases there was a larger difference as in the two gauges on Diagonal 2 shown in Figure 14. In this case the strain difference for the truck in Lane 1 was 9 με or approximately 6% of the peak value. It is important to note that in all cases, the shape of the response is similar for both gauges.

The final activity in the calibration procedure was to examine the difference in magnitude of the strain response between the measured results and the theoretical models. The desired outcome of this process is to develop a single calibration factor for the model such that all model results can be multiplied by this factor. This activity focussed on the static results for controlled loading in Lane 1. In general the calibration factor between theory and measured response was determined using the method of least squares on the difference between the two. This calibration fit was performed for all measurement points.

Two calibration fits were conducted. The first used both gauge readings on each diagonal which effectively produced a fit to the average of the gauge values shown in Figures 13 and 14. The second produced a fit to the maximum values of either gauge pair on a diagonal yielding the more conservative factor.

The global average fit produced a model calibration factor of 0.87 with a standard deviation in the difference of 11 με. The maximum value fit produced a model calibration factor of 0.93 with a standard deviation in the difference of 9 με. An illustration of the calibration fitting is shown in Figure 15 for Diagonal 8. The original B&T prediction is shown as a dashed line which is then used to create the global average fit and global maximum fit lines compared to the measured response. The least squares fit was performed on all gauges simultaneously to produce a single factor so the fitted lines will not be a perfect match to any one gauge but will minimize the error on all gauges. The average peak strain for a vehicle in Lane 1 for the all gauges is 90 με. The average difference in fit factor between 0.87 and 0.93 is therefore only 5 με.

The calibration exercise revealed that the theoretical model predicted the general behaviour of the stiffening truss elements very well and magnitude of member strain (ie. the member demand) produced by this model could be conservatively multiplied by a calibration factor of 0.93 to produce good agreement with measured response under controlled conditions. This information can now be used to update D/C predictions and LLCF predictions for the deck system.

Ambient Traffic Response

Having established the reliability of the monitoring system and performed a calibration of the theoretical model, the remaining benefit of the strain monitoring is to collect daily data on the traffic loads under actual operating conditions. Strain data is being continuously collected at a frequency of 1 Hz on all gauges for a period of one year. Unlike the controlled test results, the magnitude of the vehicle loads is unknown and will vary with traffic patterns. These results will also include the effects of the multiple vehicle presence of large and small vehicles. A sample of one week of strain data from Diagonal 3 is shown in Figure 16.

This data must be analyzed statistically. The important information is the magnitude and frequency of the peak values of strain for each gauge. Over a one year period this will produce a reasonable estimate of the maximum expected values which can then be compared against the model maximum for the multiple lane loads. The D/C and LLFC ratios can then be updated for both model calibration and traffic pattern calibration. These updated predictions will then be used as the basis for future maintenance planning on the deck system.

CONCLUSIONS

A brief summary of the procedure used to access the current performance of the deck stiffening trusses of the MacKay Bridge was presented. The methodology included rigorous modelling for the purpose of producing Demand versus Capacity Ratios and Live Load Capacity Factors that can be used for establishing the safety of the deck system against code evaluation limits. To increase the reliability of this evaluation, the theoretical work was coupled with actual strain readings from the structure under both ambient and controlled loading conditions. The theoretical model was used to identify the most critical components for monitoring. Measured response from controlled loading was used to calibrate the theoretical model. This process confirmed that the model predicted the general behaviour of the stiffening truss with respect to longitudinal and transverse truck position very well. Further comparisons demonstrated that the magnitude of strain produced by the models could be conservatively taken as over-predicting the actual member demand by only 7%. The model was therefore determined to be very reliable.

Continuous strain readings will be collected under ambient traffic for a one year period. The statistical characteristics of the bridge loads will be computed and used to update the maximum traffic load predictions for the bridge.

In both cases the use of structural monitoring has increased the reliability of the bridge evaluation process and will produce recommendations for bridge maintenance that have been calibrated based on actual response.

REFERENCES

Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-06, and Commentary, CAN/CSA-S6.1-06, Canadian Standards Association, 2006.

Table 1. Position of Test Truck during Calibration

Position ID	Distance from C/L Halifax Tower
1	36530 mm
2	46620 mm
3	50660 mm
4	55710 mm
5	60760 mm
6	65810 mm
7	69840 mm
8	74890 mm
9	79940 mm
10	84990 mm

Table 2. Comparison of Measured and Theoretical Lane Reduction Ratios

	Measured Response		Theoretical Response	
	Peak Strain ($x10^{-6}$)	Reduction Ratio	Peak Strain ($x10^{-6}$)	Reduction Ratio
Lane 1	-150	1.00	-167	1.00
Lane 2	-109	0.72	-127	0.76
Lane 3	-67	0.44	-77	0.46
Lane 4	-30	0.20	-34	0.20

Figure 1. A Murray MacKay Bridge

Figure 2. Halifax Approach Span (courtesy B&T)

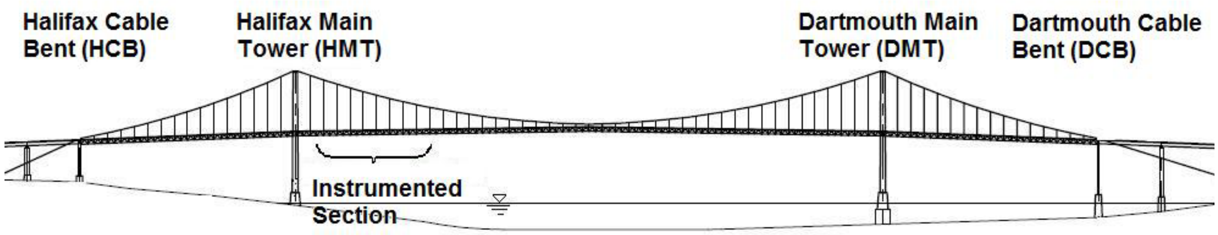

Figure 3. Profile of Suspension Bridge Section of MacKay Bridge (courtesy B&T)

Figure 4. Suspension Bridge Longitudinal Stiffening Truss prior to Addition of Maintenance Access Walkways in 2007

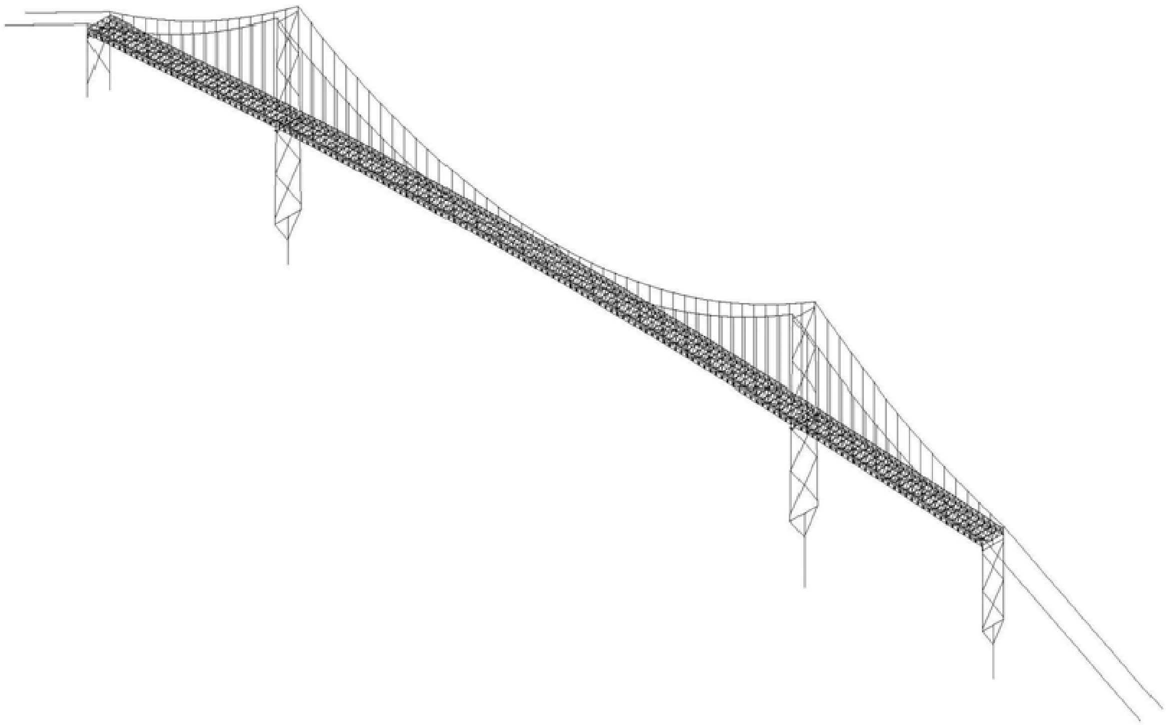

Figure 5. 3D Model of the Suspended Spans of the A. Murray MacKay

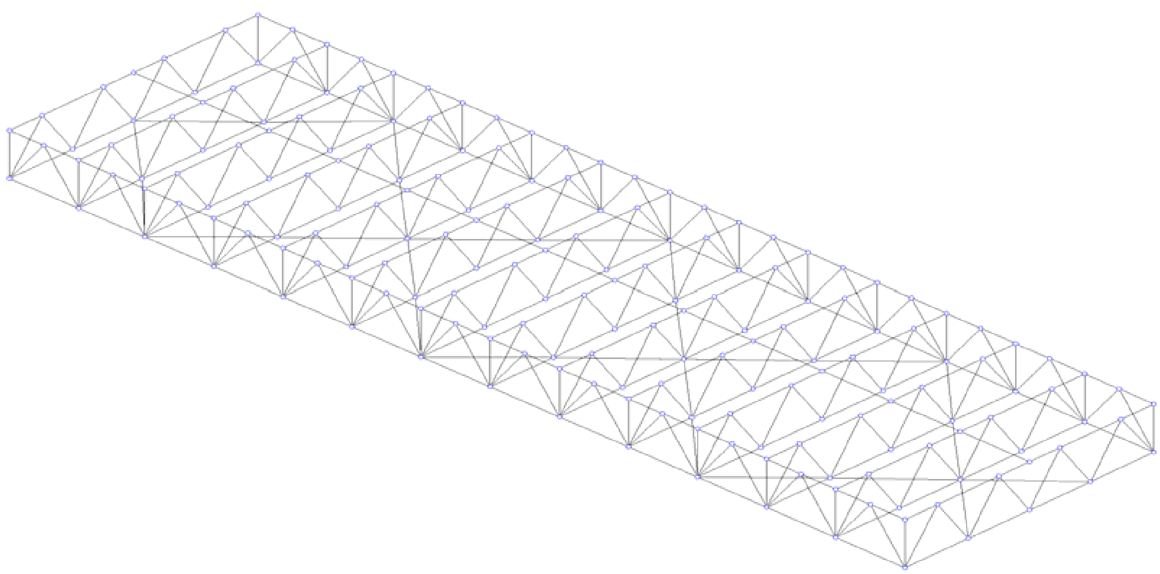

Figure 6. Deck Truss System in the Global Model

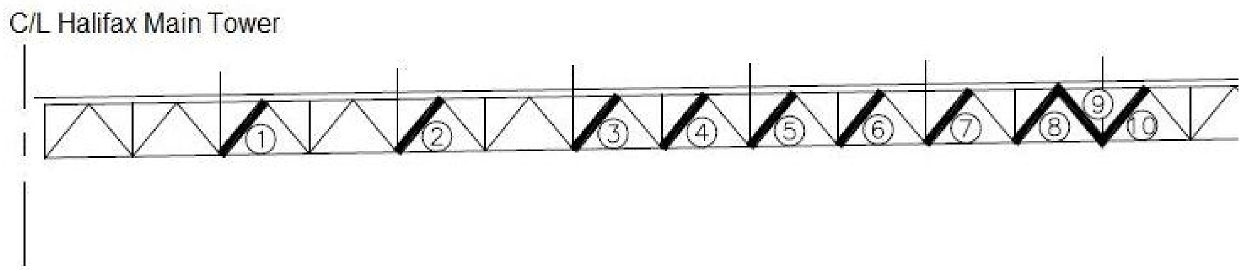

Figure 7. Location of Instrumented Diagonals

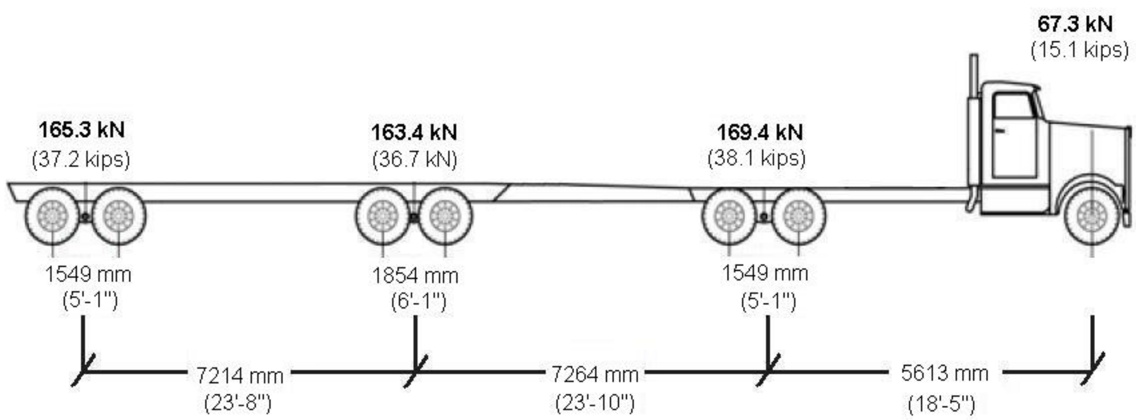

Figure 8. Details of Calibration Tests Vehicle

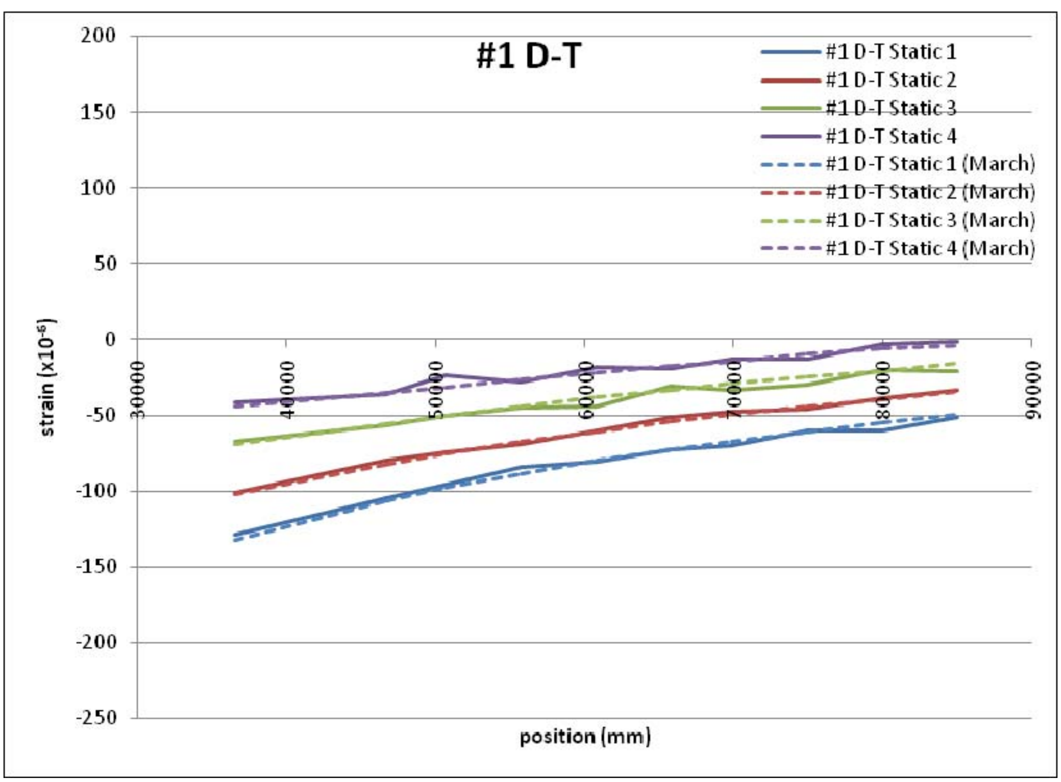

Figure 9.Comparison of Static Test Results for Gauge 1

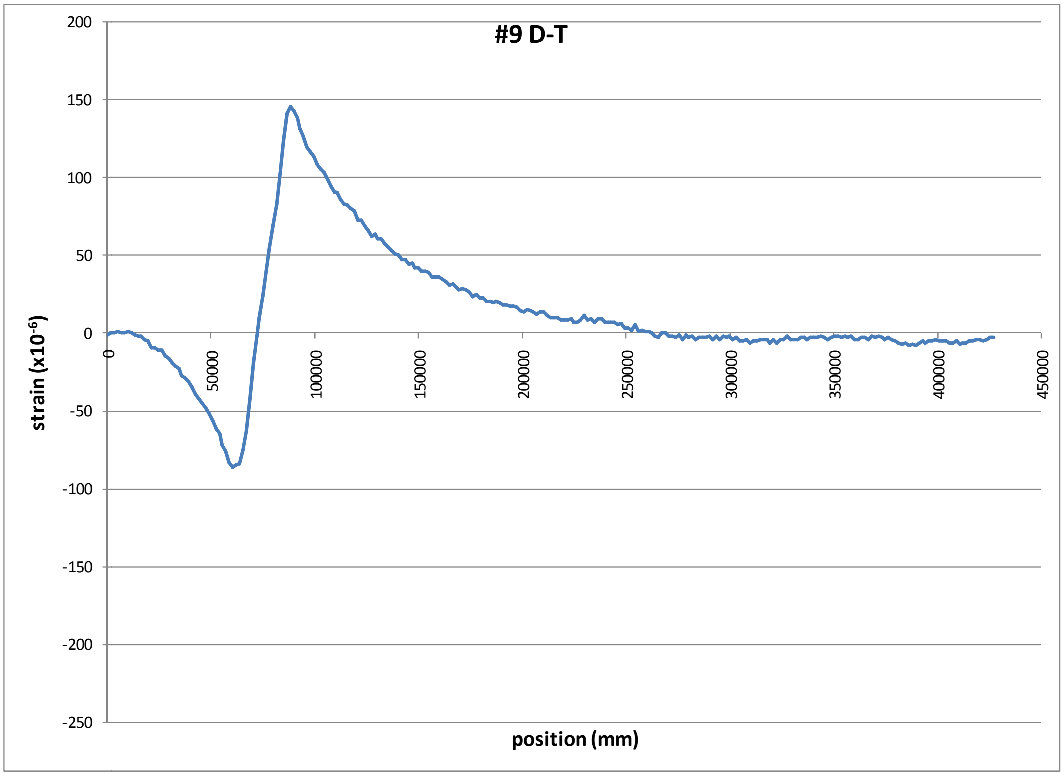

Figure 10. Response of Diagonal 9 for Slow Speed Travel in Lane 1

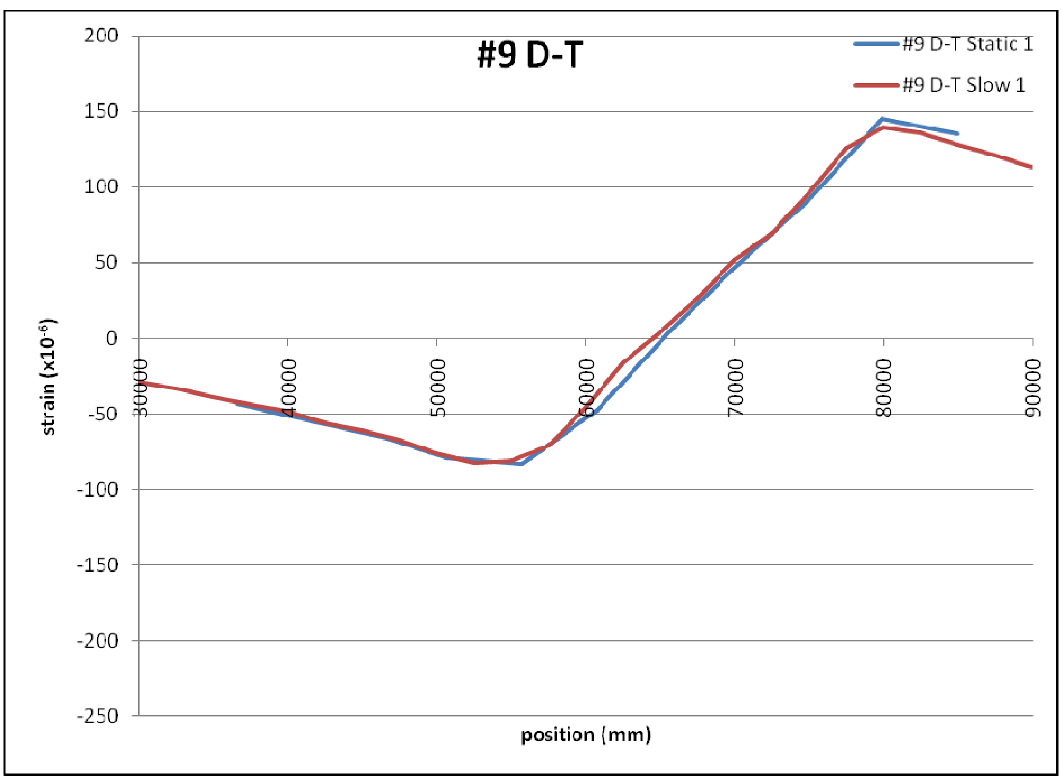

Figure 11. Comparison of Static and Slow Speed Test Data for Diagonal 9

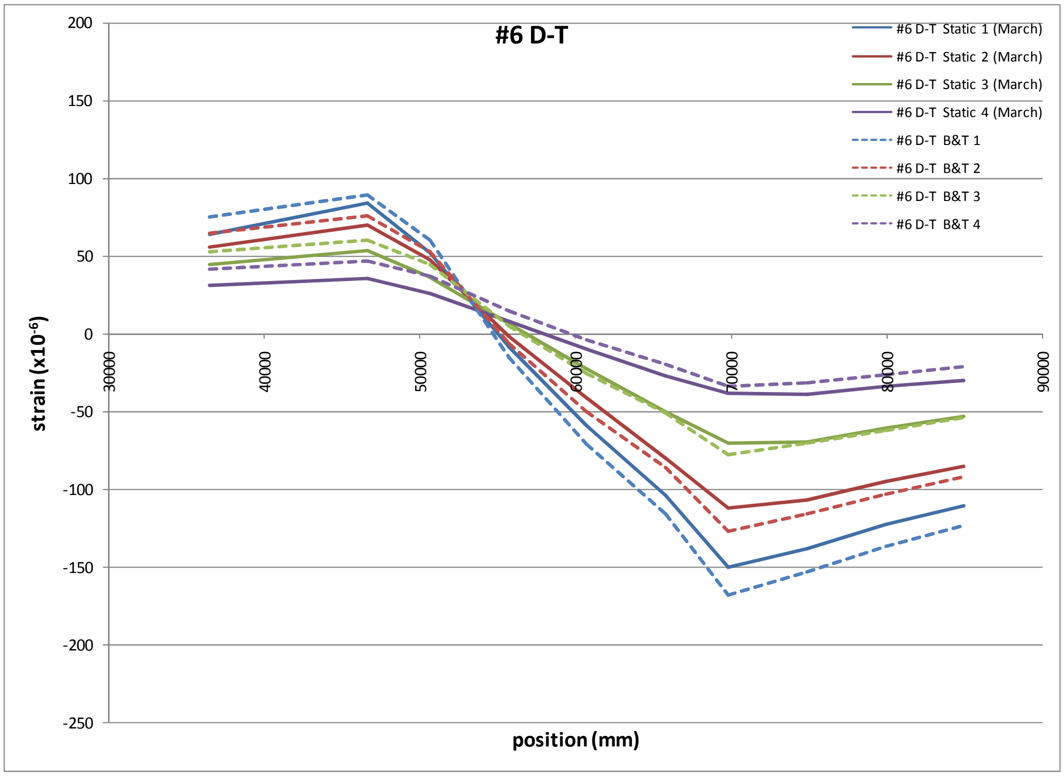

Figure 12. Comparison of Theoretical and Measured Response for Diagonal 6

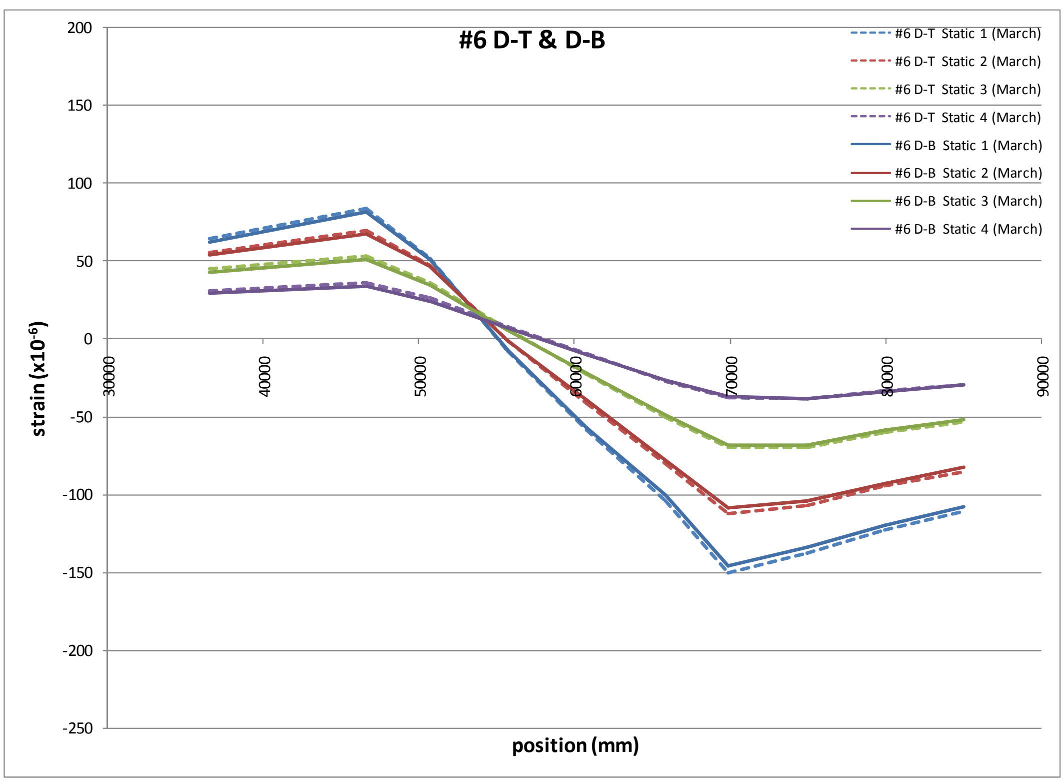

Figure 13. Comparison of Gauges on Diagonal 6

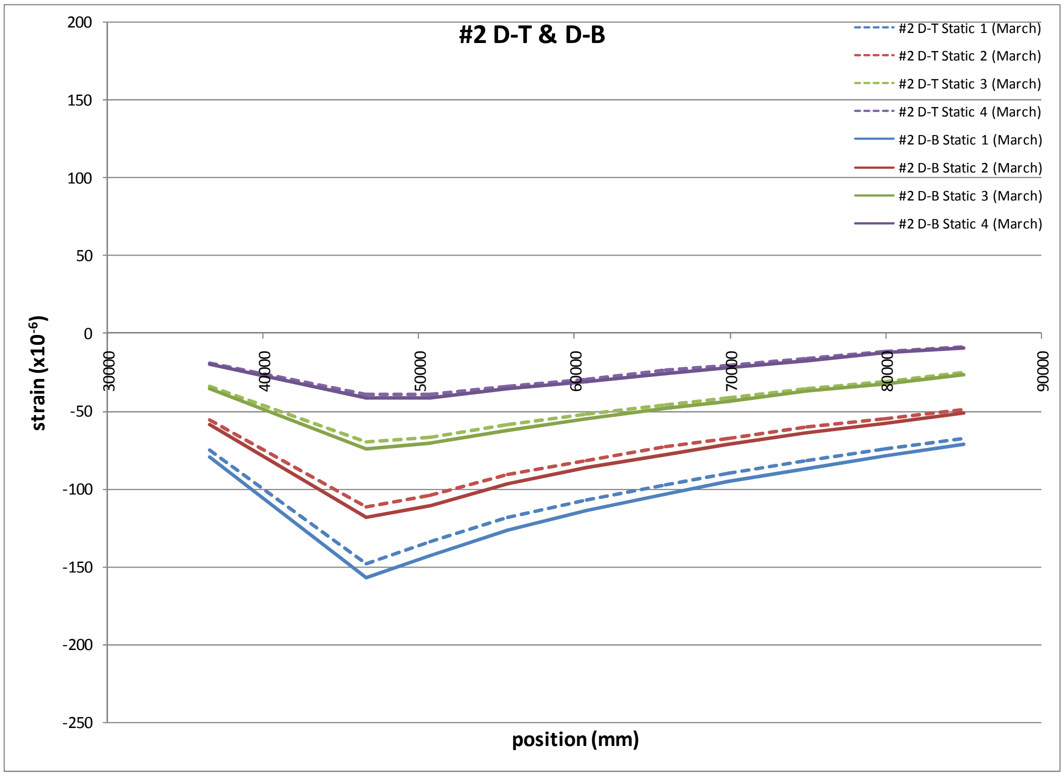

Figure 14. Comparison of Gauges on Diagonal 2

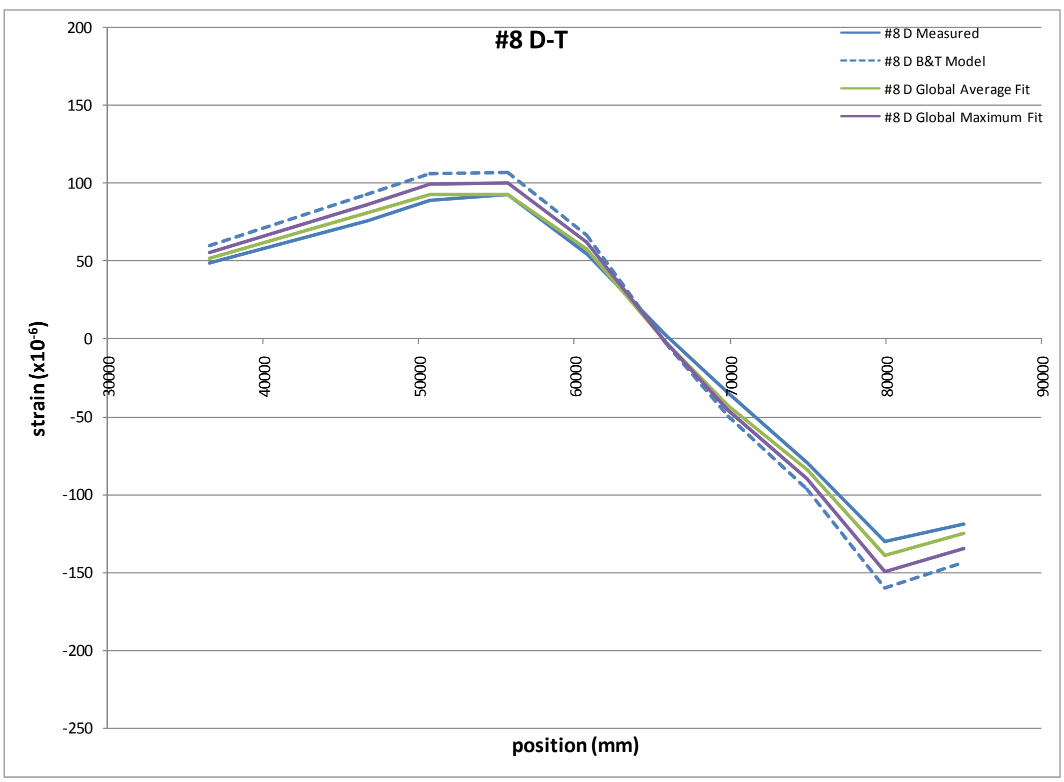

Figure 15. Example of Model Calibration for Diagonal 8

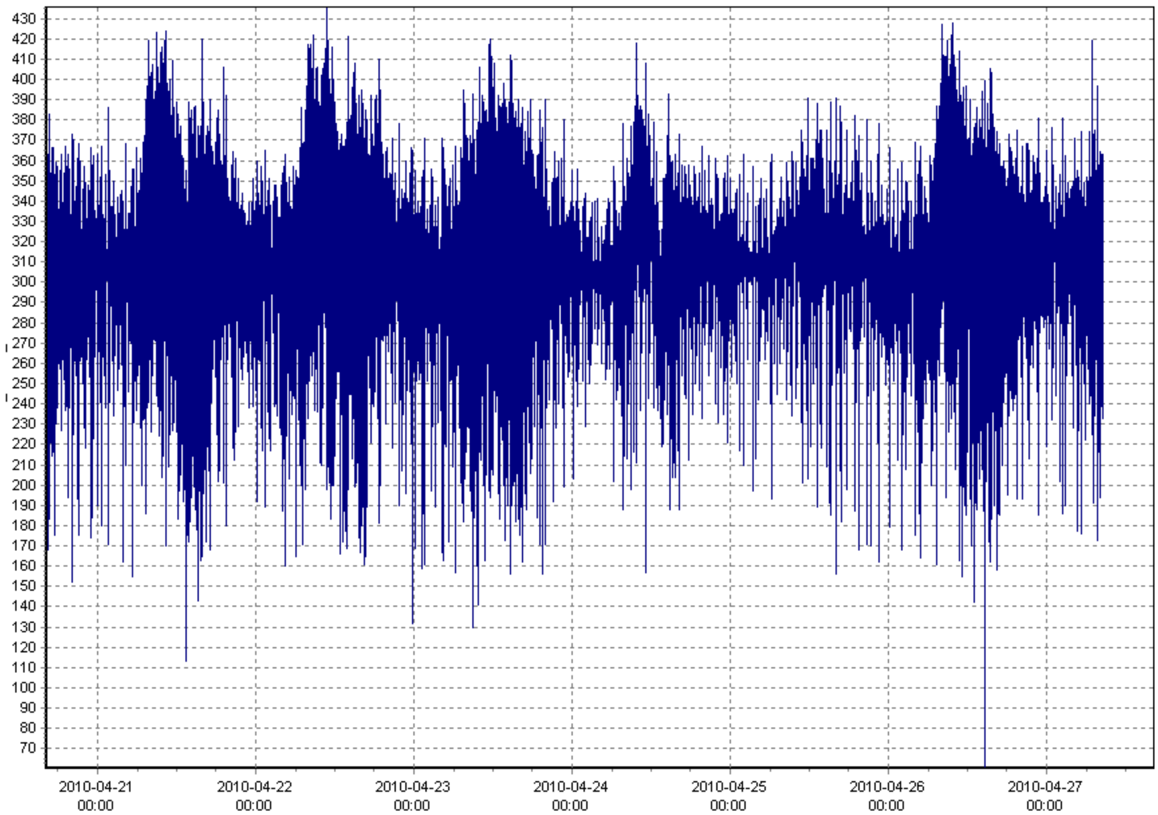

Figure 16. Example of 1 Week of Continuous Data for Diagonal 3

Figure 16. Example of 1 Week of Continuous Data for Diagonal 3