

**DEFLECTION MONITORING SYSTEMS IN STATIC AND
DYNAMIC CONDITIONS**

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1 INTRODUCTION

There are various systems commonly used to monitor the movement and overall behavior of bridges. These systems include classical methods of monitoring, such as, accelerometers, optical surveys, and base-line systems. There are also more experimental methods for monitoring the deflection of bridges, such as, Global Positioning Systems (GPS) and strain gauge systems. Most of these systems have proved reliable for providing short-term and long-term deflection data for static loads and thermal effects applied to the bridge, and for low frequency dynamic movement. However, these systems do not provide reliable data in high traffic conditions or seismic events.

The objective of this study was to develop two bridge deflection monitoring systems for use in the Kealakaha Stream Bridge to be built on the Island of Hawaii. The first system utilizes a base-line reference with Linear Variable Displacement Transducers (LVDTs) to record bridge vertical deflections during ambient traffic flow. The second system uses fiber optic strain gages to monitor curvature in the bridge, from which the deflected shape can be obtained through double integration. This system will record both low and high-frequency deformations, including seismic response of the bridge. Both systems proved effective in laboratory trials, and were successfully installed in the H3 North Halawa Valley Viaduct (NHVV) for a trial field application.

Current deflection monitoring systems are presented in Chapter 2. Development of the LVDT base-line system is presented in Chapter 3, while the fiber optic strain gage system is described in Chapter 4. Chapter 5 presents conclusions and recommendations based on the results of this study.

2 DEFLECTION MONITORING SYSTEMS

2.1 Accelerometers

Traditionally, accelerometers are the primary instruments used to monitor structural response to seismic ground shaking. They can also be used to determine the movement of a structure during dynamic motion. The acceleration history of the structure can be integrated numerically to determine the velocity trace. This in turn can be integrated to yield the deflection record. According to Çelebi and Sanli [2002], these deflections are not precise because of the numerical integration process. The nature of the signal processing requires (a) selection of filters and baseline correction (the constants of integration), and (b) use of judgment when anomalies exist in the records. Therefore, double numerical integration of the accelerometer record generally introduces a number of errors into the resulting deflection record. Furthermore, the errors are more apparent for permanent displacements, which accelerometer measurements most likely cannot recover at the centimeter level [Çelebi and Sanli, 2002]. Usually, the permanent displacements are assumed to be zero; therefore, there is error in the resulting relative displacements. Comparison of deflection records at various locations along the bridge span to determine the deformed shape is an unreliable measure of bridge deformation without the use of other equipment.

2.2 Optical Surveys

An optical survey is another term used for a direct leveling survey. A direct leveling survey is an operation that is used in determining the elevations of points, or the difference in elevations of points, on the earth's surface. The process of direct leveling is used in bridge deflection monitoring by comparing data from a survey done at the time of construction, to data gathered at a later survey. The difference in these two sets of data is the overall vertical movement of the bridge.

The process of an optical survey uses an Engineer's level, and a leveling (Philadelphia) rod, to obtain the various sets of data. Optical surveys were performed on the North Halawa Valley Viaduct (NHVV) of the H3 freeway at various intervals during and after construction. To ensure safety and accuracy the surveys were performed when the viaduct was closed to traffic. A benchmark (BM), a position with a known location and elevation, is taken as a starting point. The tripod with the Engineers level on it is set up approximately midway between the BM and the point where the elevation is to be determined (Fig 2-1).

The benchmark reading is referred to as the backsight (BS), while the second point recorded is the foresight (FS). After taking readings the tripod is moved to a point between the FS and the next location where the elevation is desired.

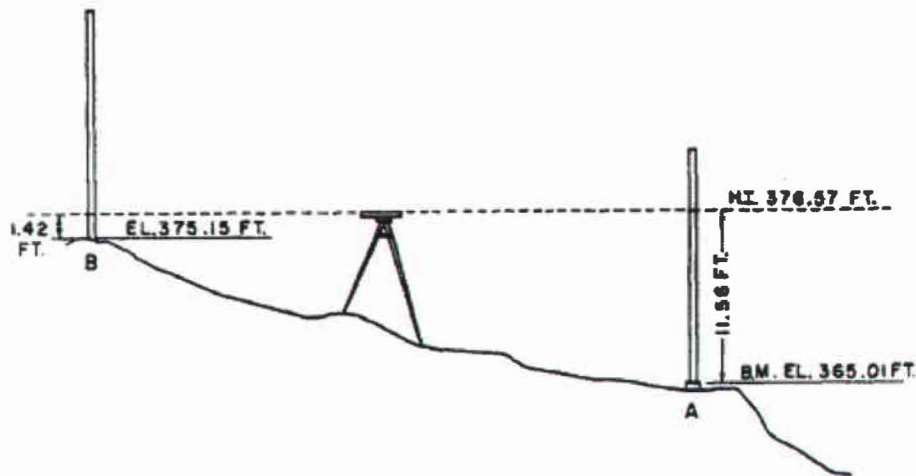


Figure 2-1: Procedure for direct leveling

It's important to locate the tripod midway between the FS and BS because it corrects for instrument error, and errors due to curvature and refraction. The FS then becomes the backsight and the next point becomes the foresight. This procedure is repeated for the entire distance being surveyed (Fig 2-2). In order to correct for errors, the run should close at another benchmark or return to the original benchmark. Any small discrepancy between the survey results and the ending benchmark are generally distributed proportionately to all the elevations along the survey run.

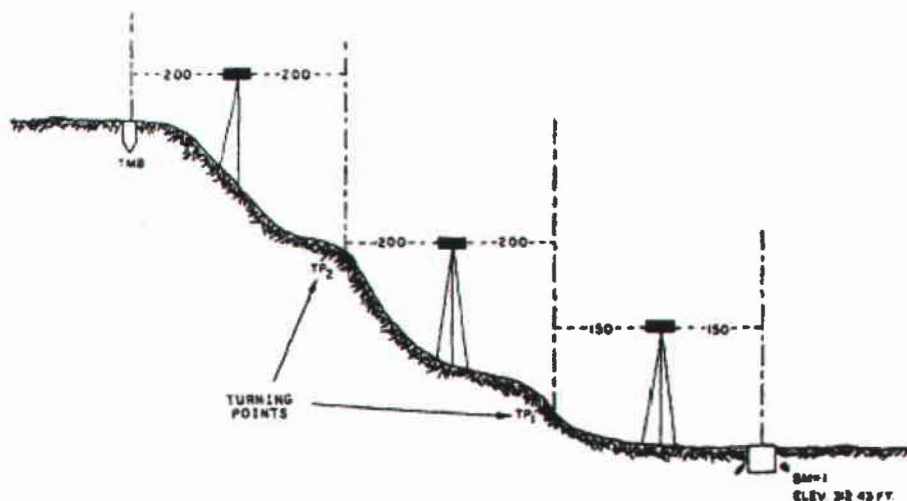


Figure 2-2: Entire run for an optical survey

An optical survey of Unit 2IB of the NHVV was performed in March of 1995 by the State surveyor after completion of construction on this Unit. The State surveyor has performed subsequent surveys approximately every three years. The difference between these later surveys and the original survey represents the total vertical deflection of the viaduct including any foundation settlement or pier shortening.

Optical surveys have a number of disadvantages, mainly the labor involved in performing a survey. In order to perform an optical survey at least two people are needed, preferably three people should be used. Human error is another important disadvantage of an optical survey. If the survey extends past mid day, the temperature effects on the bridge can affect the readings. Due to all these factors optical surveys of bridges are usually only performed as part of an annual inspection program..

One advantage of an optical survey is that it can provide a reliable long term verification of other deflection systems.

2.3 Base-Line System

2.3.1 H3 Base-Line system

A taut-wire base-line system consists of a high-strength piano wire fixed at one end of a bridge span, strung over a pulley at the other end of the span, and attached to a heavy weight (Fig 2-3) [Lee and Robertson, 1995]. The vertical movement of the bridge girder relative to the ends of the span is recorded by measuring changes between the base-line and girder. A base line system was used to monitor span deformations for 4 spans of Unit 2IB of the H3 North Halawa Valley Viaduct (NHVV). The four

instrumented spans were from piers P8 to P9, P9 to P10, P11 to P12, and piers P12 to P13. This base-line system is best suited for monitoring deformation under static and long-term movement of the bridge.

Figure 2-3 shows the base line setup. At the dead end of the base-line system the #8 piano wire is firmly fixed to the dead end bracket by clamping between two metal plates. At the live end, the piano wire passes over a pulley and supports an 80lb weight to provide a constant tension in the wire. The weights are attached to safety chains, which are attached to the wall, in case the wire fails. The piano wires are coated with Linseed oil to prevent corrosion. Since the viaduct is curved the end brackets were positioned such that the wire came as close as possible to the centerline of the box girder at midspan. The points of deflection to be recorded were located at alternate construction joints throughout the span. Steel base plates were installed at each of these locations using anchor bolts. Three small angle guides were welded to each base plate to ensure consistent positioning of the reading caliper.

A modified Mitutoyo digital caliper with an 8 in. range was used to take the measurement readings. The caliper was modified by attaching a magnetic base with an on/off switch to one end of the caliper. Once the magnet is secured to the base plate the caliper is adjusted until the free end is just touching the wire. The reading represents the relative distance between the baseline and the top slab of the viaduct. The difference between an initial reading and subsequent readings is the girder deflection relative to the ends of the span. Measurements are generally taken early in the morning to reduce the influence of thermal effects and traffic flow on the bridge.

Just as in the case of the optical survey the base-line system has its drawbacks. One major disadvantage is the need for manual monitoring and the lack of continuous readings at all stations simultaneously.

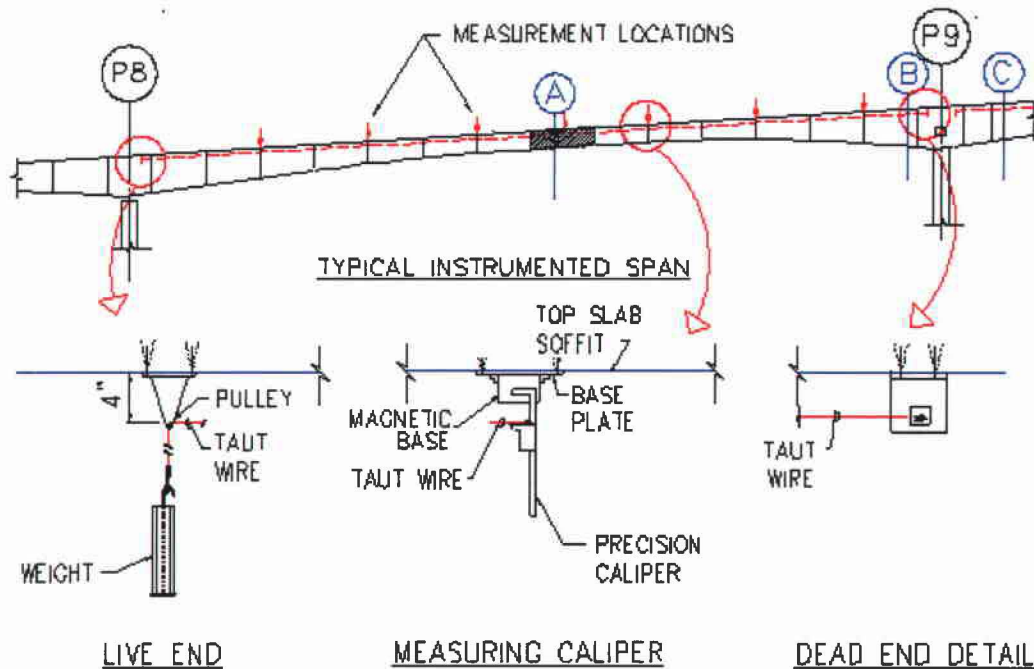


Figure 2-3: Base-line system using a digital caliper

2.3.2 Weighted Stretched Wire System

A modification of the base-line system is called the Weighted Stretched Wire System (WSWS). The same principle is used in which a weighted wire passes over a pulley at one end and attached to a fixed point at the other end of the bridge span (Figure 2-4). To measure the deflection in the wire a displacement transducer is used, such as an LVDT [Stanton, 2002]. The LVDT allows for continuous monitoring of the movement of the structure.

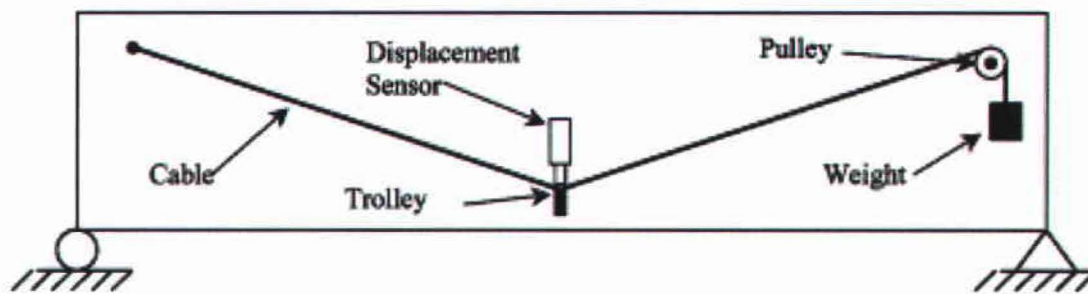


Figure 2-4: Weighted Stretched Wire System (WSWS) (Stanton, 2002)

The plunger (rod) for the LVDT rests on top of a Trolley that is supported by the stretched wire. The weight of the trolley is determined to produce a constant sag in the wire. Since the pulley is assumed frictionless, the sag in the wire should remain constant due to the effect of static equilibrium, regardless of the movement of the girder or the wire. The relative fixed reference frame of the trolley to the attachment points allows the vertical deflection of the girder at the trolley location to be measured directly. This system allows for the continuous monitoring of the girder vertical deflection during short-term events such as during thermal movement and ambient traffic, as well as for long-term effects such as creep, and shrinkage and prestress loss. The system is not suitable for seismic or other high frequency dynamic events because the various components of the system (piano-wire, weights, etc.) will vibrate during the event, thus distorting the deflection measurements.

3 MODIFIED BASE-LINE SYSTEMS

3.1 Multiple LVDT Base-Line system tested in the laboratory

In order to provide continuous monitoring at multiple points along a girder span, an LVDT base-line system was developed. This system was first used during the bending test of a prestressed concrete beam as part of another research project underway in the structural testing laboratory at UH.

After successful performance in laboratory conditions, the system was installed on a temporary basis in the H3 NHVV to evaluate its performance under field conditions.

3.2 Laboratory Applications

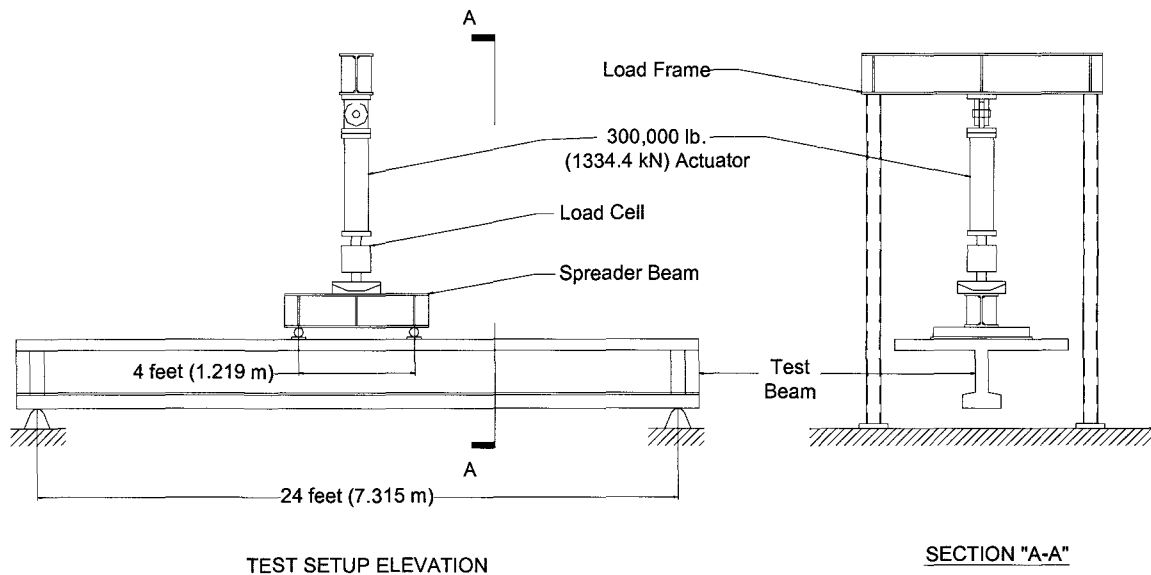


Figure 3-1: Test setup for Pre-stressed girder beam

The pre-stressed pre-cast concrete 24-foot girder with a 5' wide by 4.5 in. reinforced concrete top slab was tested under four point loading using a 300,000 lb load frame (Figure 3-1).

The types of displacement sensors chosen for this system were Linear Variable Differential Transducers (LVDTs). An LVDT is an electromechanical transducer used to measure various types of displacements. Seven LVDTs were used as part of the baseline deflection system. Three additional LVDTs were connected to another data acquisition system and set up along the top of the slab. The first was placed at midspan, while the other two LVDTs were placed between midspan and the end support. A dial gauge was used to measure movement at the support (Figure 3-2).

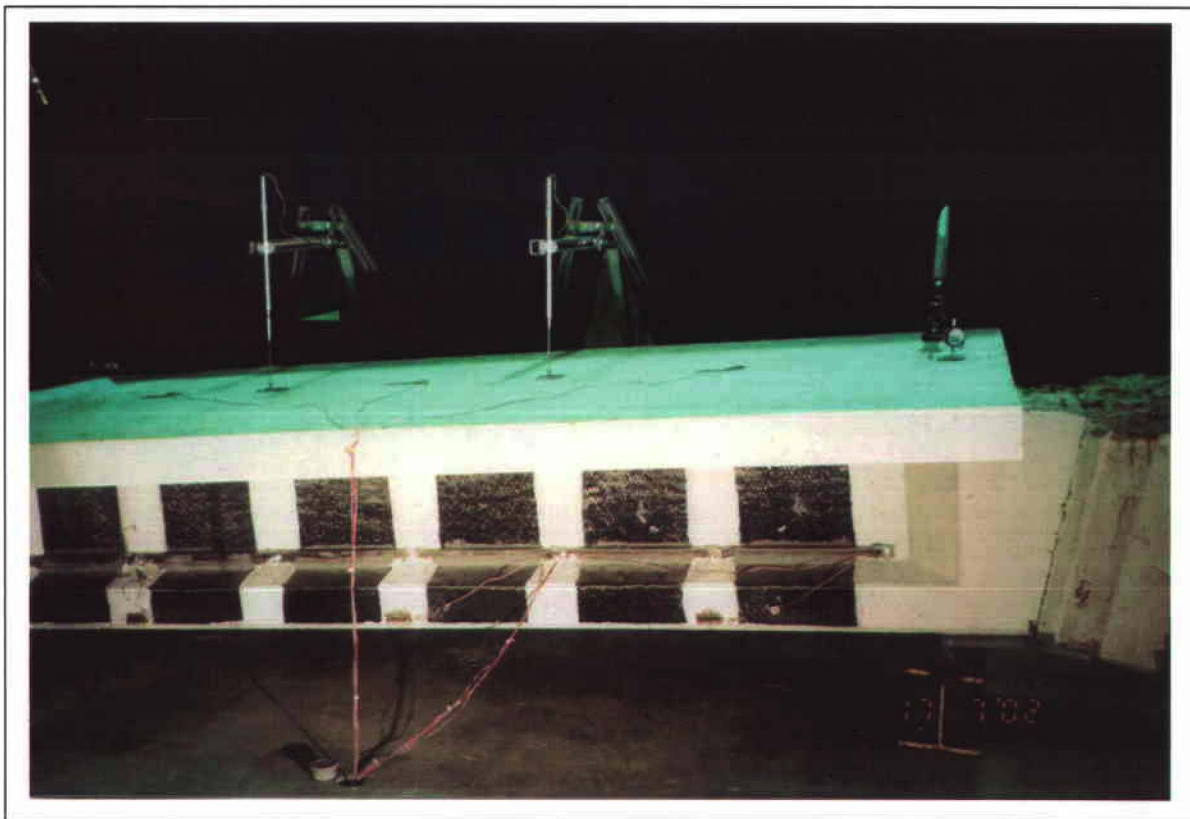


Figure 3-2: LVDTs and dial gauge placed on the top slab

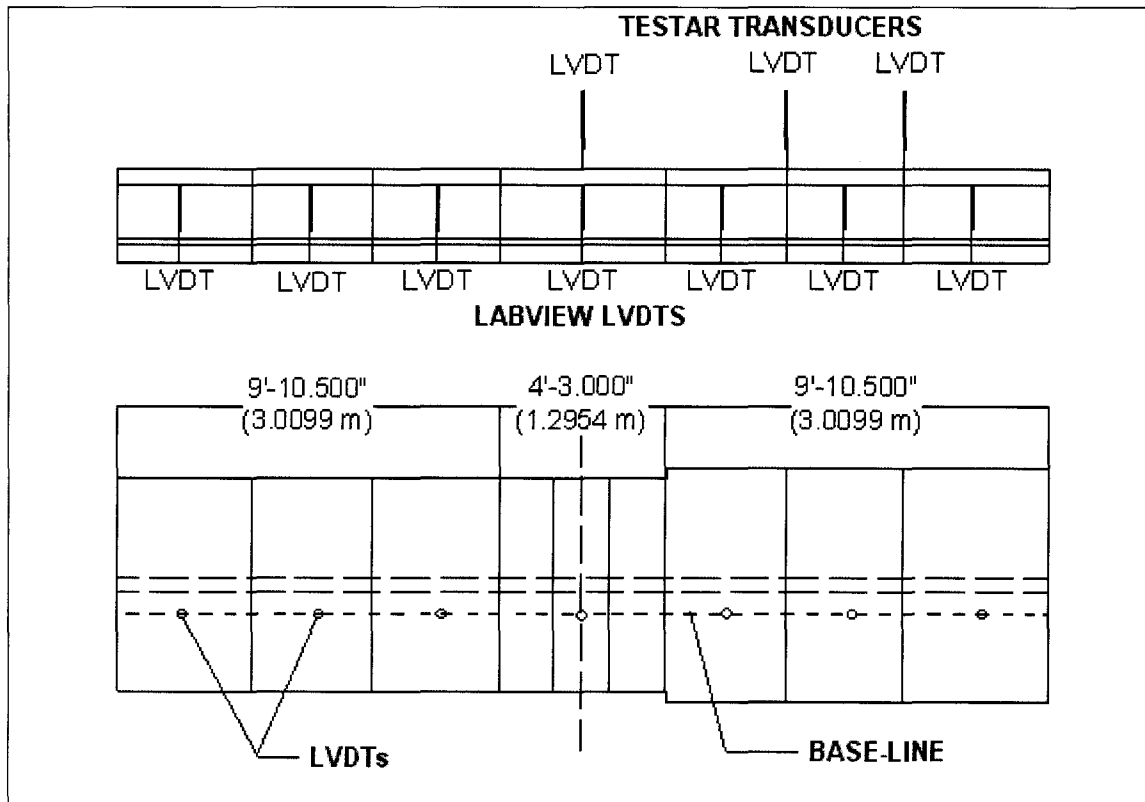


Figure 3-3: LVDT Setup for Prestressed Girder beam

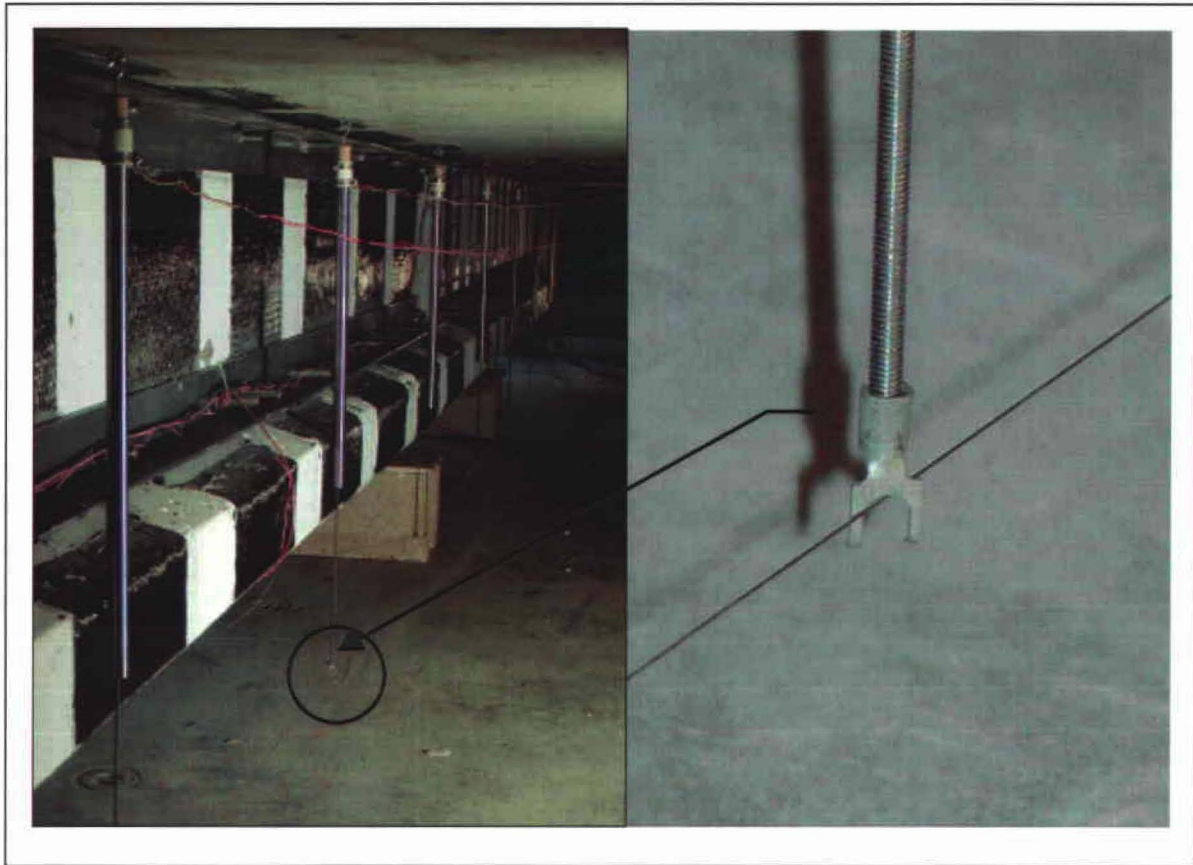


Figure 3-4: LVDT Base-Line system, Piano wire connection

The LVDTs used in this system were Macrosensors PR-750 series LVDTs, with a range of ± 5.00 in. (127 mm). National instruments products, LabVIEW and its corresponding SCXI signal conditioning technology, were used to record LVDT data at 100 readings per second during the test (Figure 3-5). To confirm the LVDT baseline system, the resulting deflected shape was compared with LVDTs located on the centerline of the top slab as part of the original beam instrumentation.



Figure 3-5: LabVIEW data acquisition system

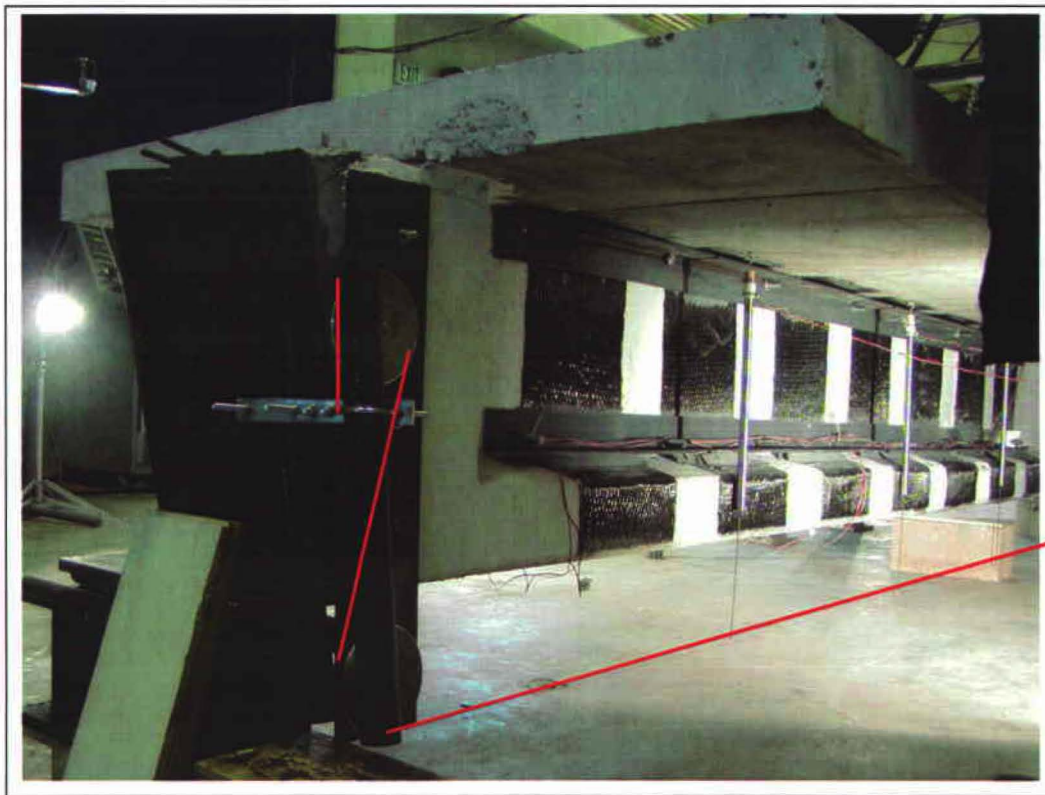


Figure 3-6: End plate connection for LVDT base-line system

Figure 3-6 shows the modified live end of the base-line piano wire system. The end plate consisted of two pulleys attached to a plate at the live end of the beam. The base-line passed from the fixed end plate, under the LVDTs, under and around the lower pulley, then over and around the upper pulley as shown by the red line in Figure 3-6. The line was then attached to a 40lb weight.

The LVDTs were attached to the bottom of the slab using plastic anchors and threaded eyebolts (Figure 3-7). A hole was pre-drilled into the bottom of the top slab about an inch into the concrete, and then a plastic anchor was fitted into the hole. The threaded eyebolts were inserted into the plastic anchor and tightened.

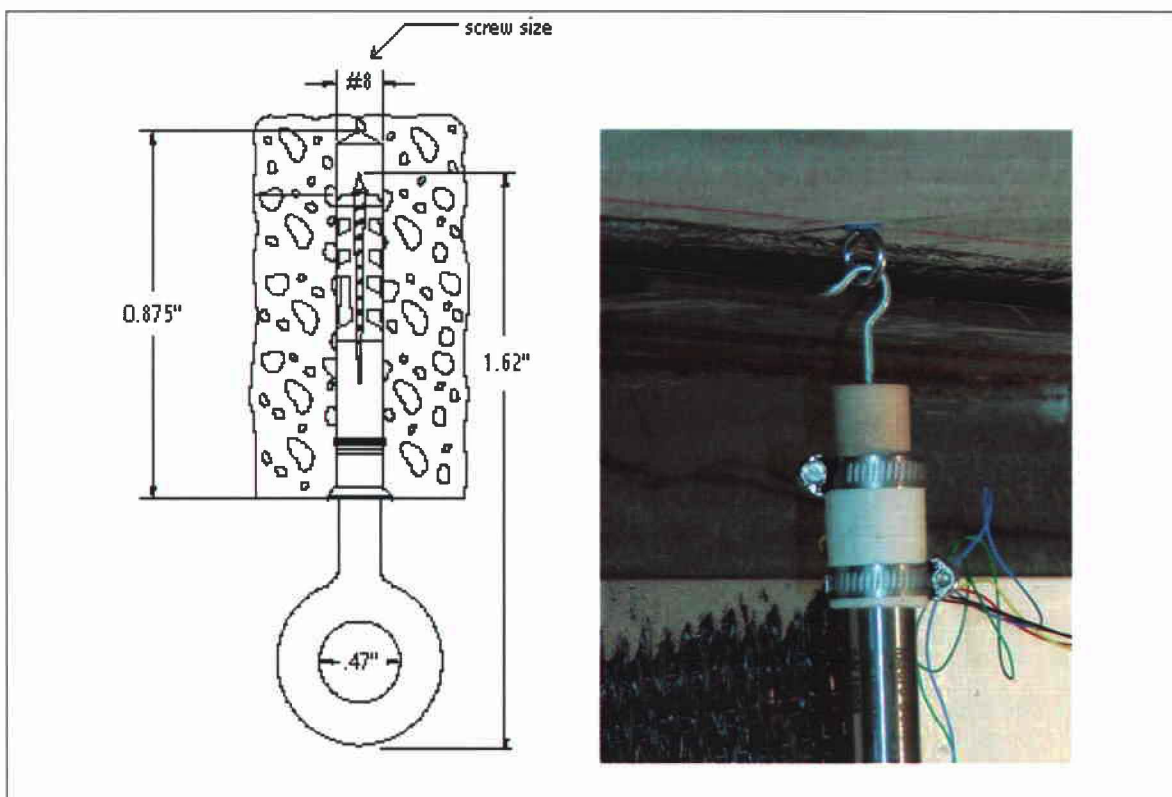


Figure 3-7: Top slab connection

A $\frac{3}{4}$ in diameter PVC pipe and a $\frac{3}{4}$ in. wooden dowel were used to make the link system on the transducer. The dowel was cut into 1- $\frac{1}{2}$ in. long sections to fit inside the

PVC pipe and a metal hook was inserted in the top of each dowel. The PVC pipe was slit along one side and two steel hose clamps were used to tighten the pipe as a clamp connecting the dowel to the top of the LVDT (Figure 3-7). The LVDTs on the top slab were connected to the data acquisition in the TESTSTAR II servo controller.

3.2.1 Results

Data were recorded from the LVDT baseline system using LabVIEW at a rate of 100 readings per LVDT per second. The failure of the beam occurred at approximately 133 kips. Figure 3-8 shows the deflected shape recorded at every 10 kip load increment.

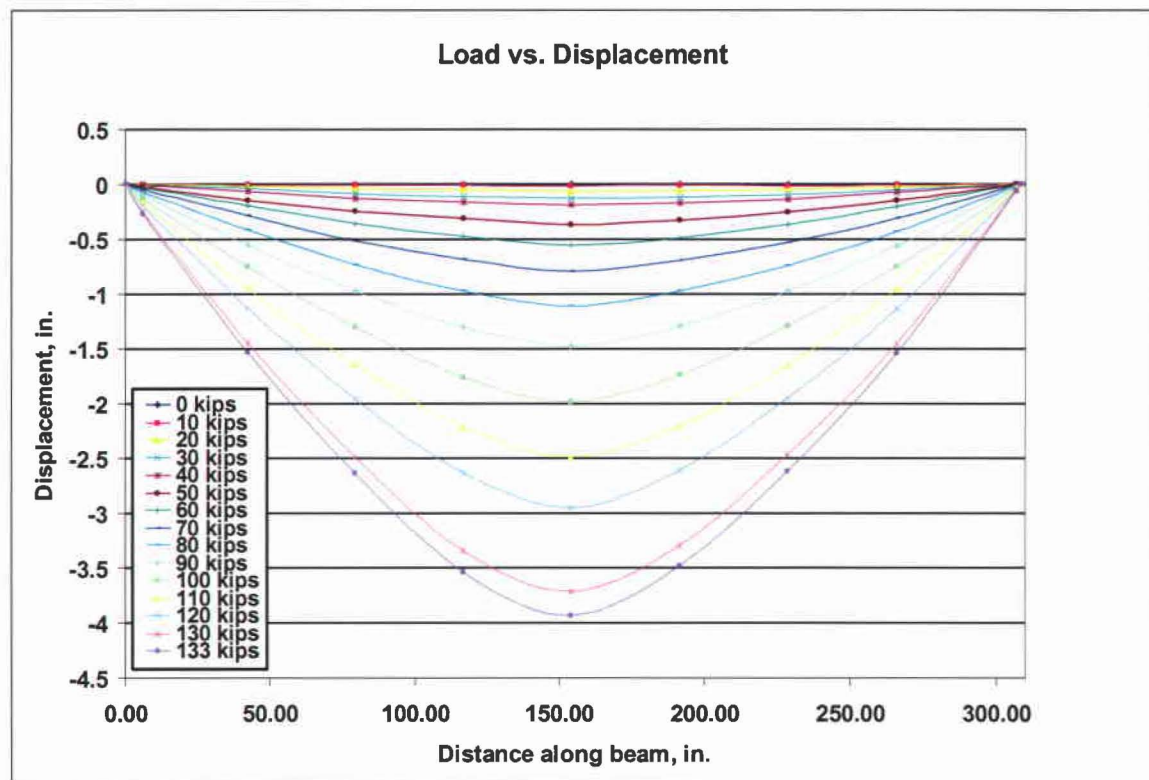


Figure 3-8: Span displacement measured by LVDT base-line system

Figure 3-9 shows a comparison between the LVDT base-line system and the reference LVDTs on the top slab at 100 Kip load. The deflected shape measured by the top slab LVDTs on half of the beam was assumed to be symmetrical about midspan.

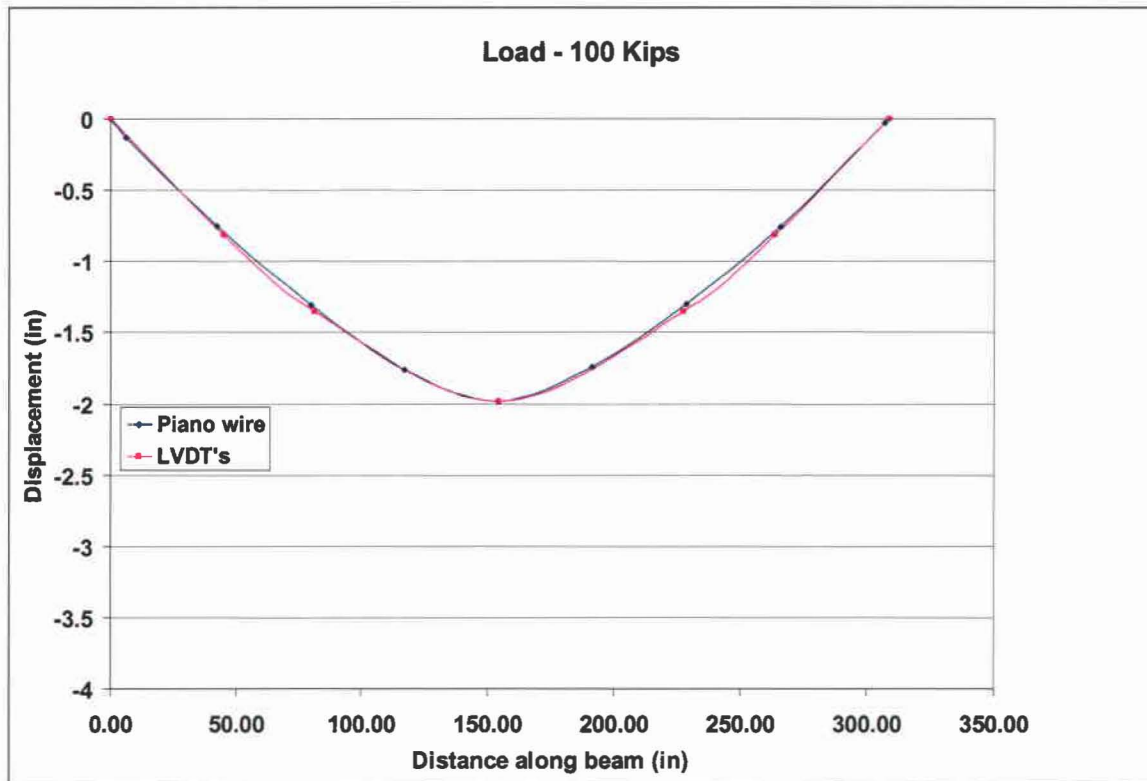


Figure 3-9: Corrected displacement at 100 Kips

The LVDT base-line system was able to provide an accurate record of the beam deflected shape. Graphs of the other loading stages during the beam test are included in Appendix A.

3.3 Multiple LVDT Base-Line system field applications

3.3.1 Description of the LVDT Base-Line system used in the NHVV

The piano wire base-line system already installed in between piers P8 and P9 of Unit 2IB of the NHVV was modified to replace the seven digital caliper reading points with seven LVDTs. The transducers were installed in the same way as used on the T-beam in the Structures laboratory test. However, the weight of the LVDT cores and threaded rods caused significant deflection of the piano wire. Turnbuckle extensions were added to the link between the LVDT and the underside of the top slab to adjust for this deflection in the piano wire.

3.3.2 Effects due to traffic loading

The LabVIEW data acquisition system was set up to record data from each LVDT twice every second. The data were collected at weekly intervals for processing at UH. Figure 3-10 shows the displacement of all seven LVDTs as a number of vehicles pass over the box girder. As a vehicle crosses the adjacent span, the instrumented span rises slightly. As the vehicle moves onto the instrumented span the displacement goes down. As the vehicle exits off the span, the deflection returns to the original value. There are three clearly identified vehicles in Figure 3-10.

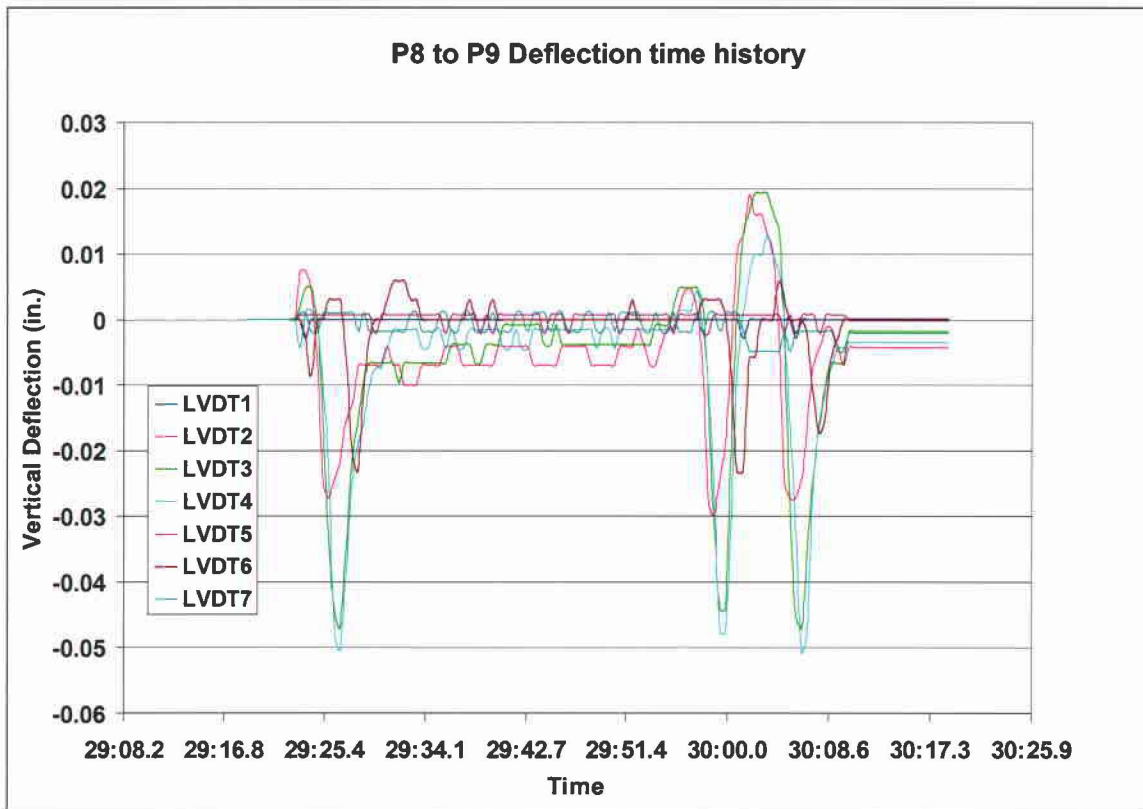


Figure 3-10: Adjusted LVDT base-line deflections

The LVDT baseline displacements were significantly smaller than anticipated based on a prior load test of the instrumented span. It was concluded that the weight of the LVDT cores was too large for the #8 piano wire used in the original baseline system. The initial piano wire was removed and a #20 wire was installed. The end weights were increased to 200 lbs to keep adequate tension in the piano wire. The LVDTs were placed on the wire and the data collected for another week.

Figure 3-11 shows that the baseline system with thicker piano wire now records displacements of about 0.2 to 0.25" for truck loads. This is similar to the anticipated deflection. The accuracy of the system could be verified by means of a calibrated truck passing over the viaduct at various speeds.

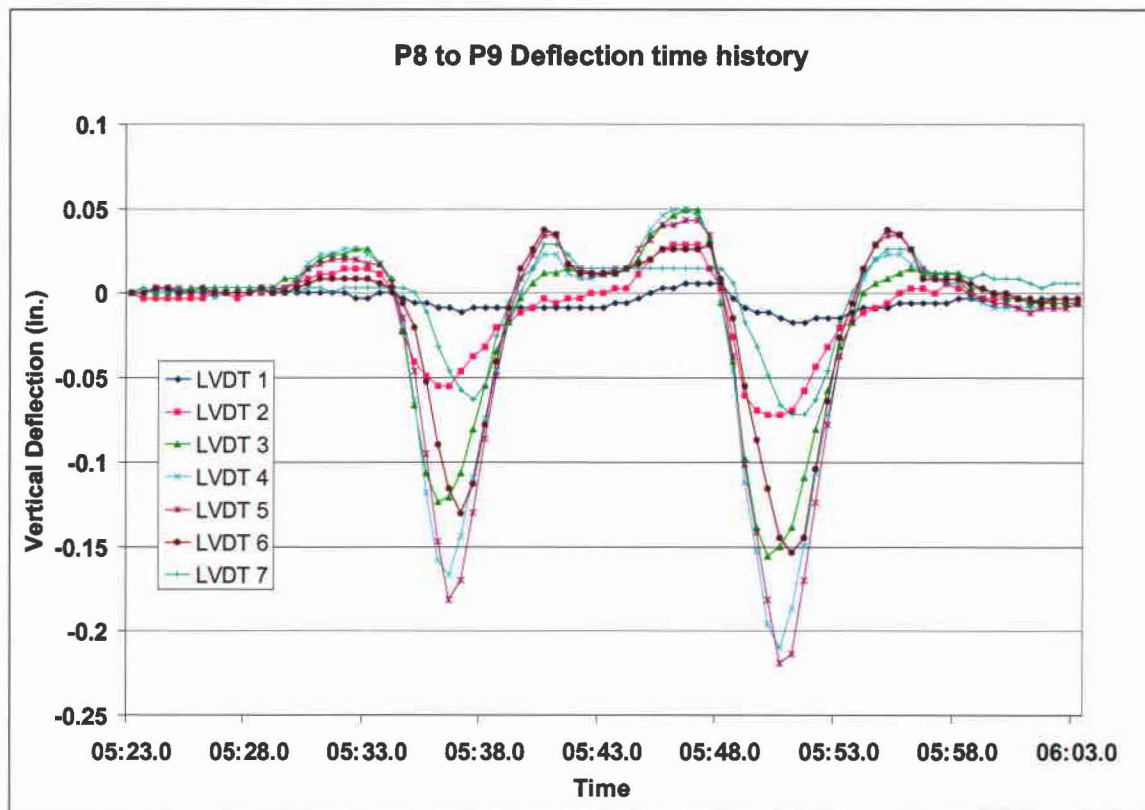


Figure 3-11: Adjusted LVDT values with new piano wire

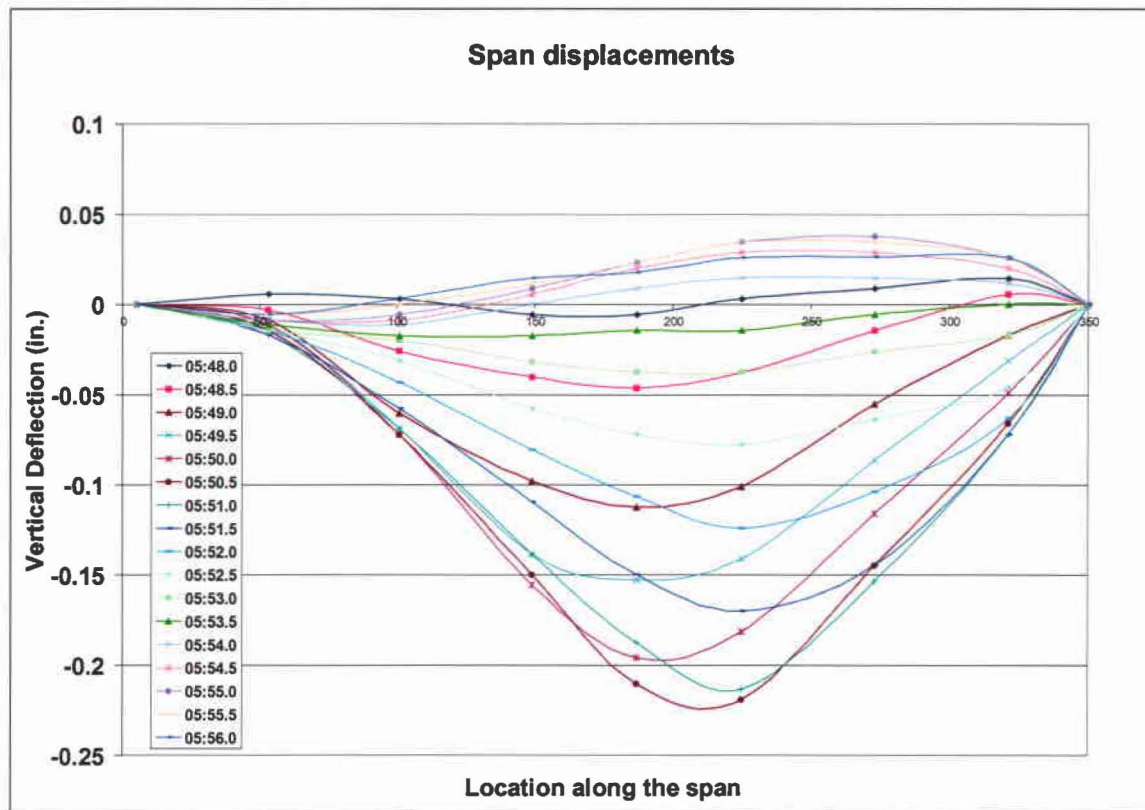


Figure 3-12: Deflection of the span due to a vehicle load

The following figures show the movement of the beam as a vehicle enters the span, drives over the middle and exits on the following span. The data is extrapolated from the graph in Figure 3-11. The figures are split up into two graphs showing the vertical deflection of the girder from piers P8 to P9. The first graph, Figure 3-13, shows the vehicle as it enters the span from P8 to P9 from the previous span. The graph shows the vehicle moving along the girder with the coinciding deflections up until mid-span.

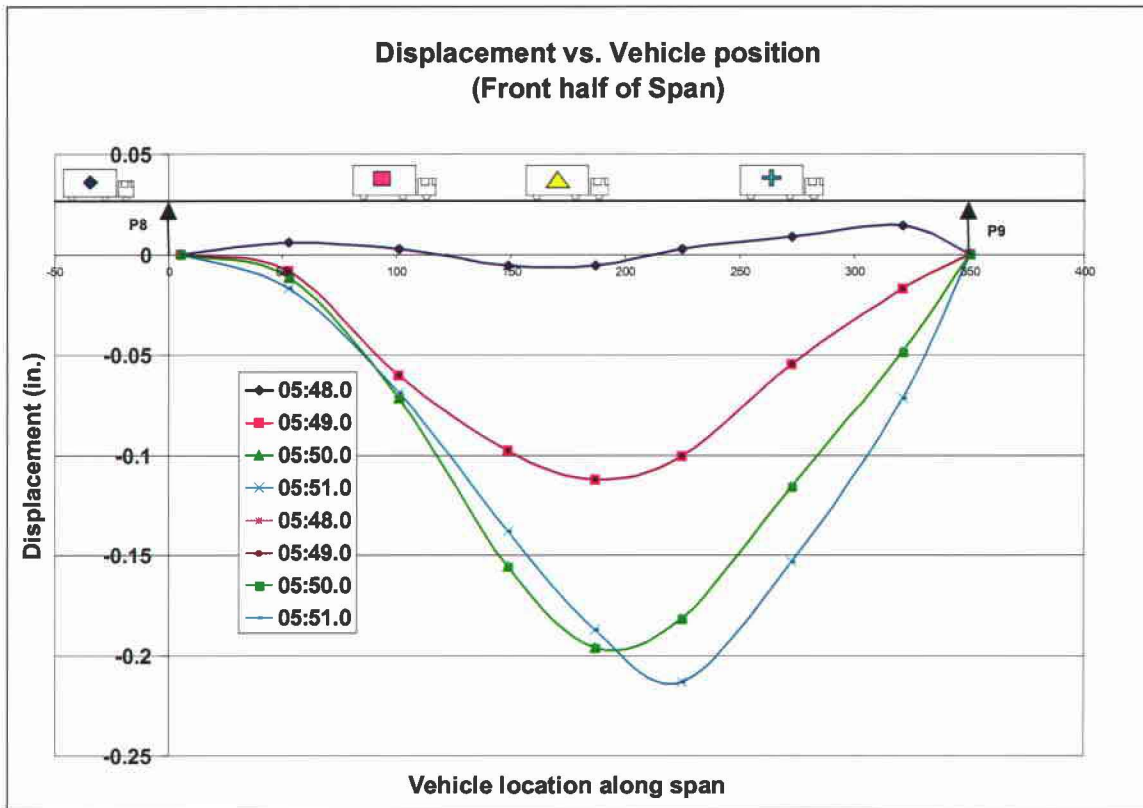


Figure 3-13: Vertical deflection on the front half of the span due to vehicle loading

Figure 3-14 shows the vehicle as it travels along the back half of the span and on to the next span. In both figures the plot line markings for the vertical deflections match the markings on the location of the vehicle.

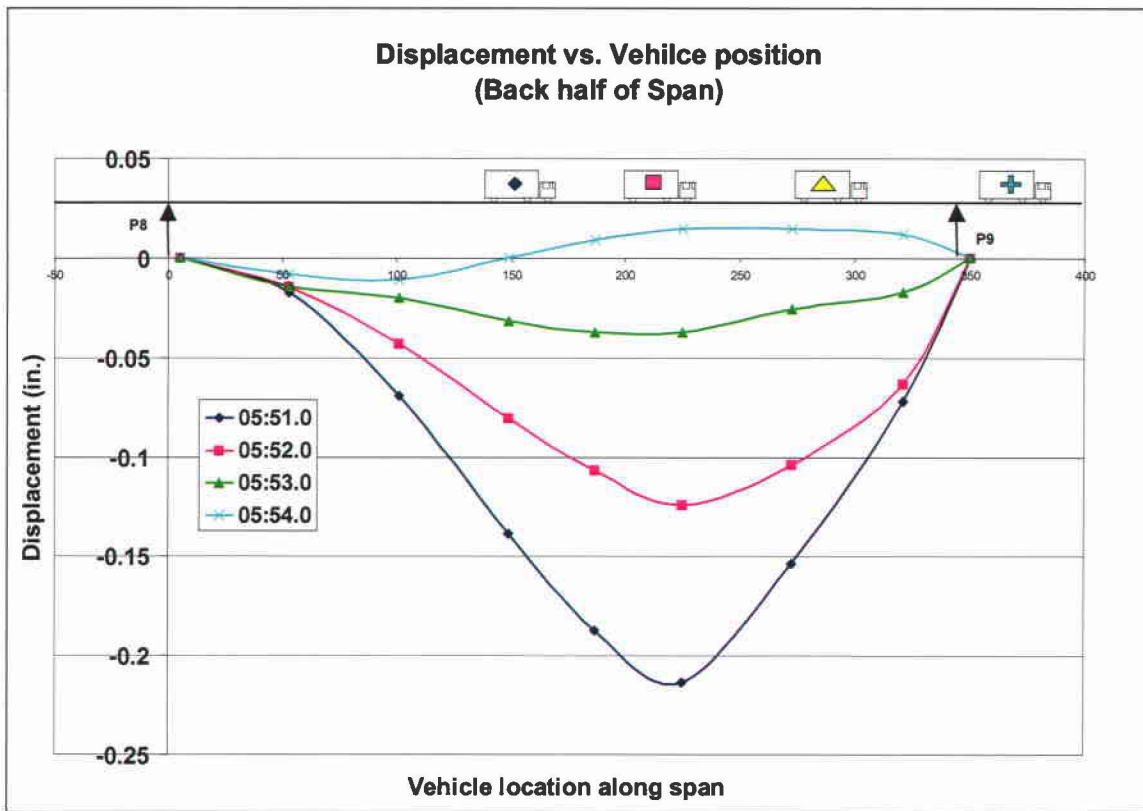


Figure 3-14: Vertical deflection on the back half of the span

3.3.3 Temperature effects

An earlier study was performed on the thermal response of the NHVV Unit of 2IB [Ao and Robertson, 1999]. Thermocouple measurements in the bridge cross-section showed that daily solar radiation resulted in a significant thermal gradient across the box girder top slab, with little change in temperature for the rest of the cross-section.

Figure 3-15 shows the analytically predicted deflections of the Unit 2IB spans from pier 7 to pier 13 due to a 10° temperature gradient through the top slab [Ao and Robertson, 1999].

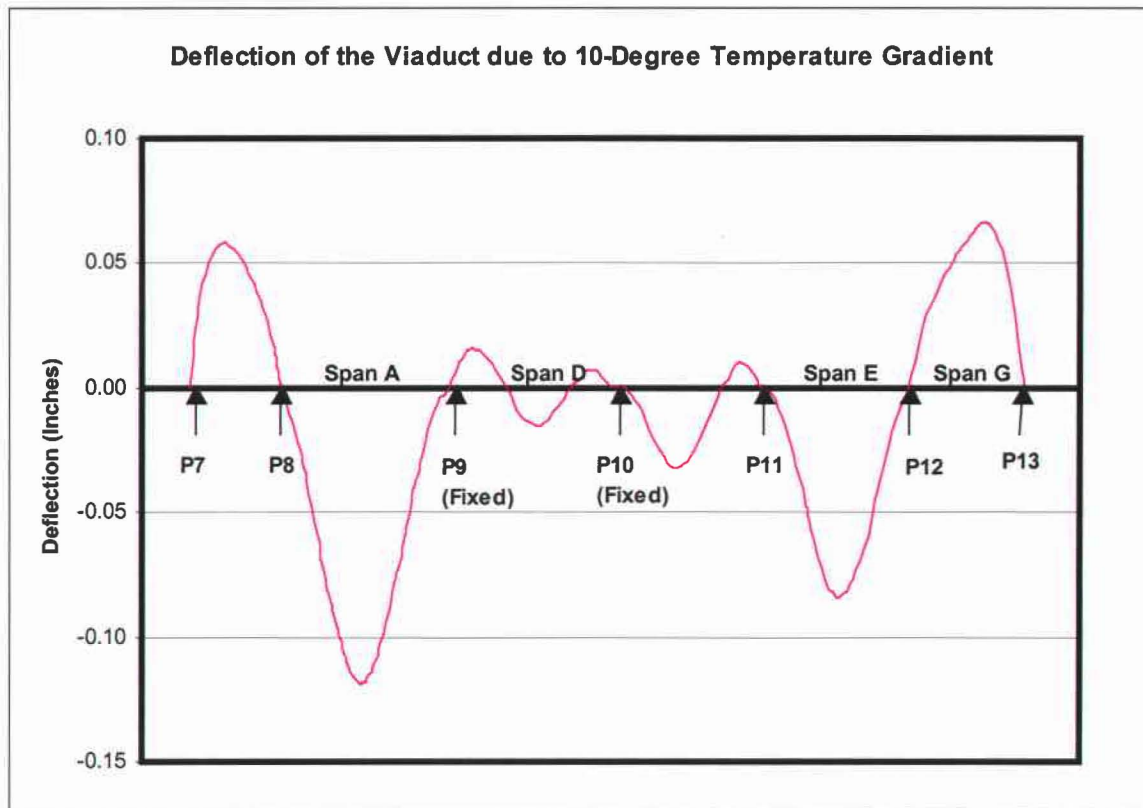


Figure 3-15: Vertical deflection of Unit 2IB due to Temperature effects

[Ao and Robertson, 1999]

As shown in Figure 3-15, the temperature gradient through the top slab is predicted to cause the girder between piers 8 and 9 to deflect downwards. This behavior was confirmed by field measurements using the original base line system with manual digital caliper readings (Figure 3-16). The deflected shape was also confirmed by support rotations measured by the tilt meters at each support (Figure 3-16).

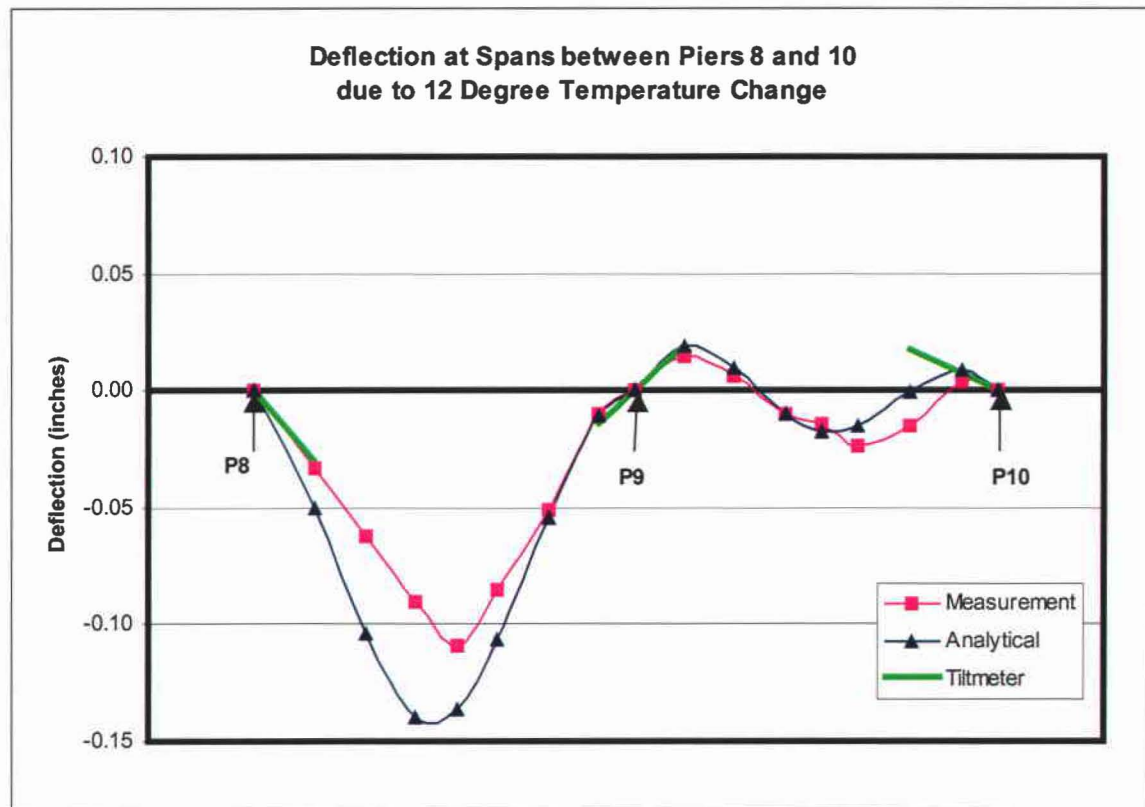


Figure 3-16: Girder deflections from Piers 8 to 10 due to Temperature gradient

(Ao and Robertson, 1999)

The previous work done was a starting point to follow for the work being done in this thesis. The information collected from the viaduct was sorted to see if the baseline system would also agree with the phenomenon of the previous tests.

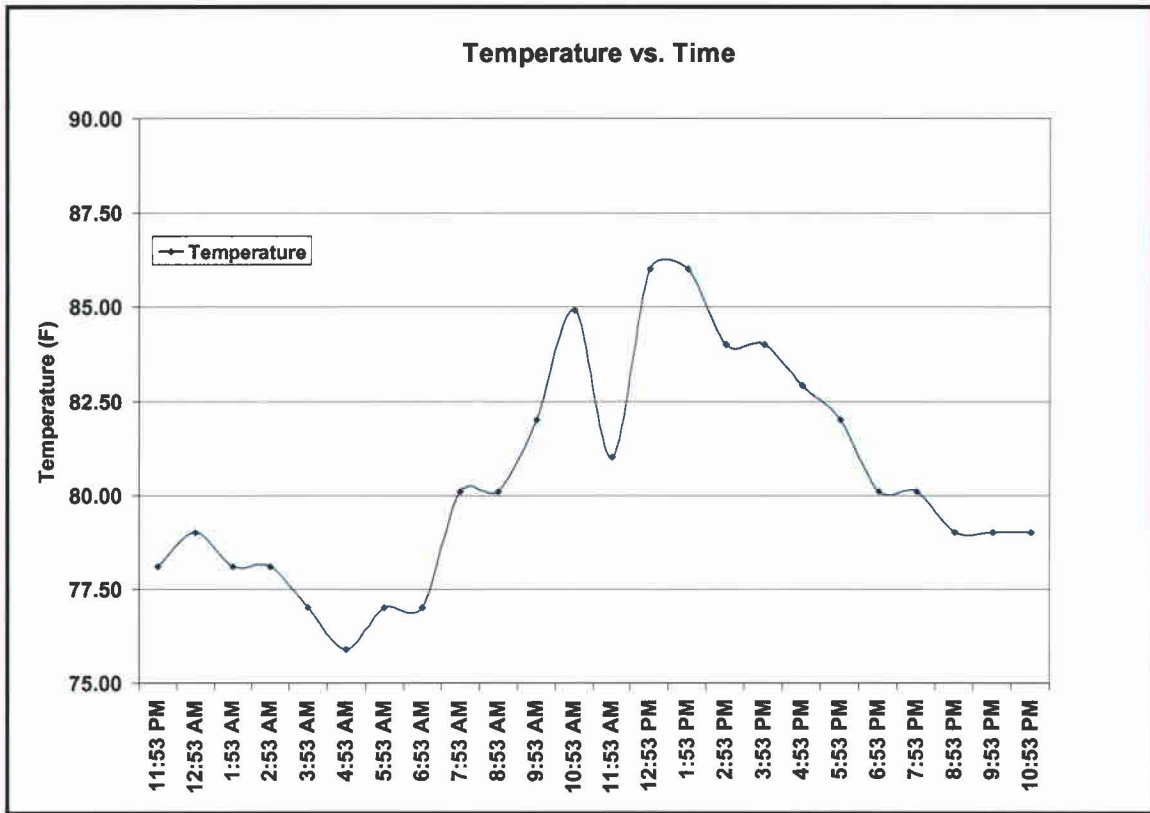


Figure 3-17: Temperature values at H3

Figure 3-17 shows the ambient temperature applied to the top slab throughout the course of a day. This is followed by Figure 3-18, which shows the displacement of the mid-span LVDT with the same time interval. The figure shows that during the evening and early morning the box girder displaces upwards. As the temperature rises towards mid day and early afternoon the box girder deflects downward.

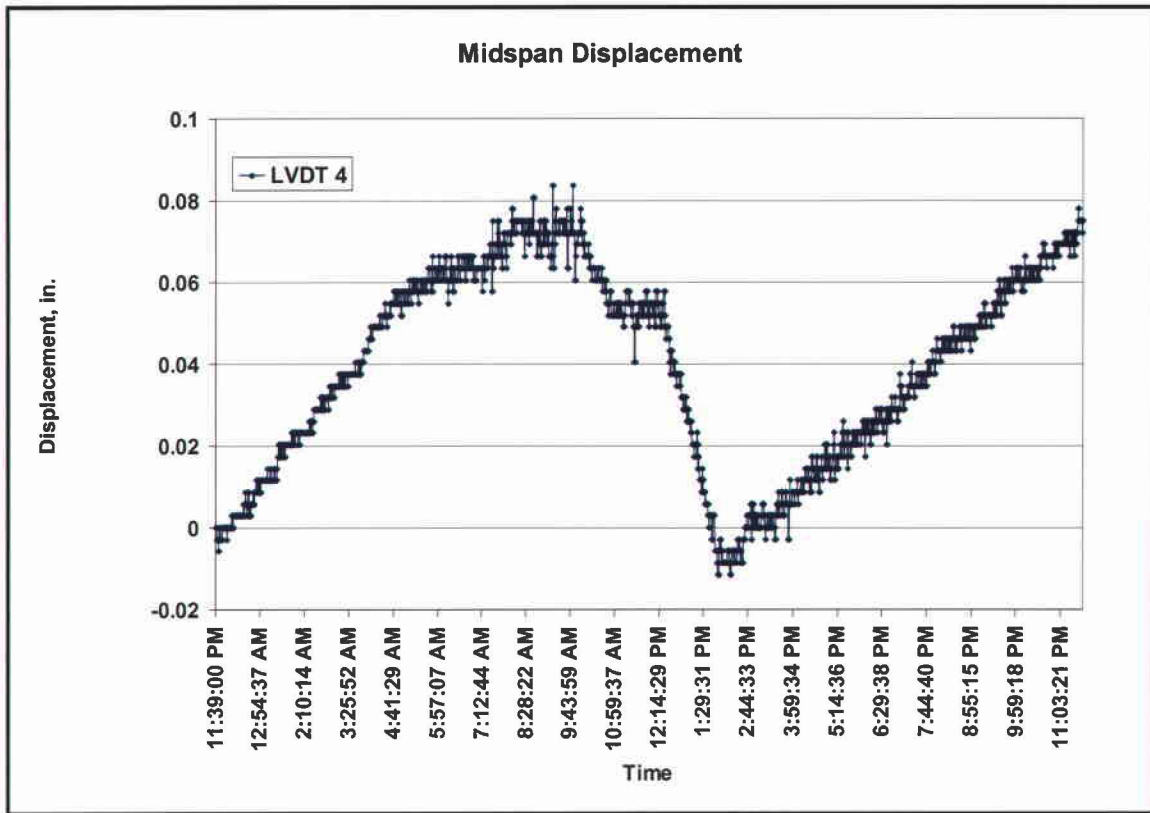


Figure 3-18: Mid-span displacement throughout the day

Figure 3-14 shows the comparison between the previous study and the LVDT base-line system. The base-line LVDT plot (blue line) was graphed using data for a ten degree ambient temperature shift, while the values for the Analytical and previously measured data were graphed using a 10 degree gradient change in the top slab. The graph does show that the girder from piers P8 to P9 behaves as expected according to the previous study.

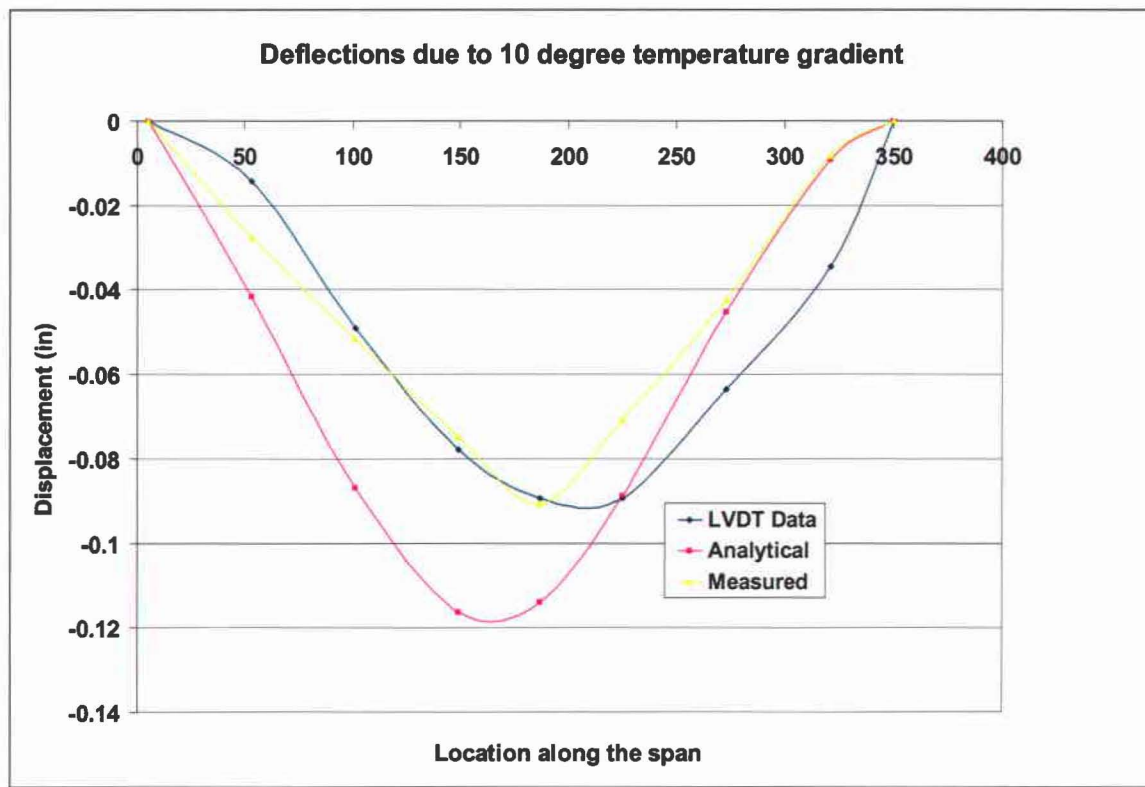


Figure 3-19: Comparison of previous Temperature study to Base-line system

4 MODIFIED STRAIN GAUGE SYSTEM

4.1 Previous Strain Gauge study

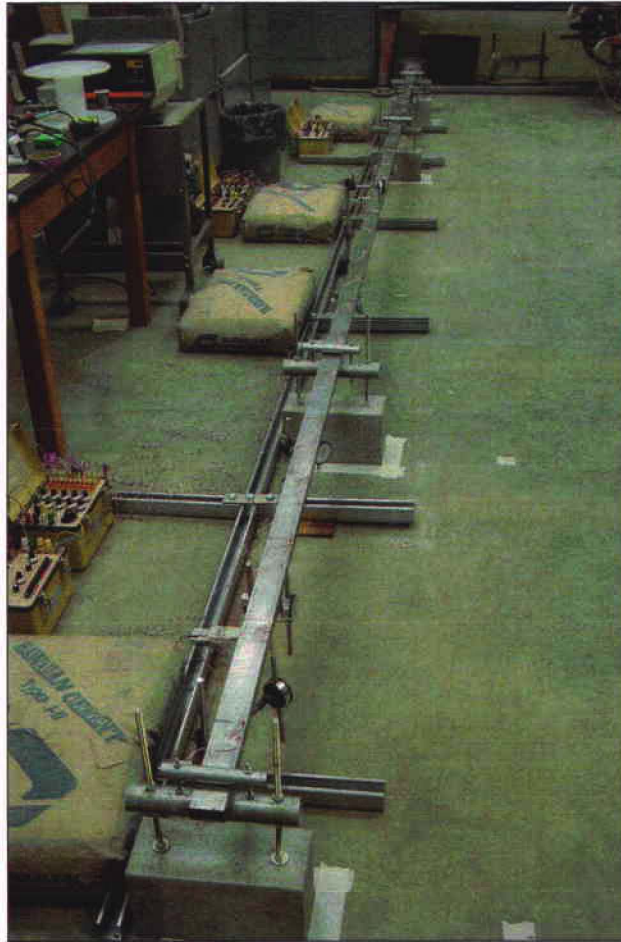


Figure 4-1: 20ft long rectangular steel tube

4.1.1 *Static Testing of Strain Gauges*

The previous study [Fung et al, 2002] was conducted on a twenty-foot long rectangular steel tube with nominal cross-section dimensions of 2 in (5.080 cm) x 1.0 in (2.54 cm) x $\frac{1}{16}$ in (0.1588 cm) wall thickness (see Figure 4-2 for exact dimensions). The beam rests on four pinned supports with a 1:2:1 length ratio for the three spans (Figs. 5.7

and 5.8). This beam approximates a 1/37-scale model of the Kealakaha Stream Bridge, but has a constant cross-section as opposed to the variable box girder section proposed for the bridge.

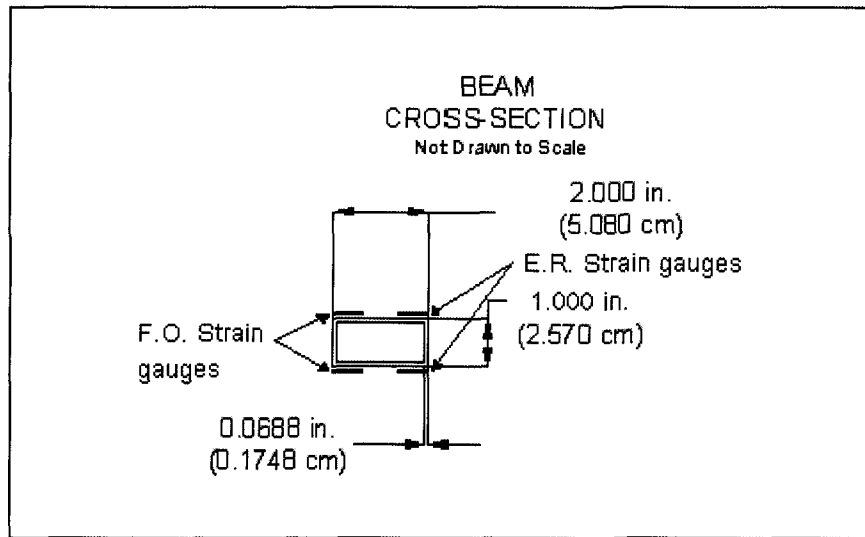


Figure 4-2: Steel tube beam dimensions with Strain gauge placements

The three-span beam was divided into four sections. Each 59 in (149.9 cm) long section was divided into three cells, requiring a placement of twelve strain gauges, one at the center of each cell (Figure 4-4). For the previous tests and for this study the configuration is referred to as the “ideal 12” strain gauge layout. Twelve Linear Variable Displacement Transducers were to serve as the “measured” deflected values, previously recorded by dial gauges. Nine LVDTs were placed at the locations of the original dial gauges (Figure 4-3). Two more were placed at support 2 and support 3, the interior supports. The LVDTs and strain gauge readings were recorded using National Instruments data acquisition system running LabVIEW.

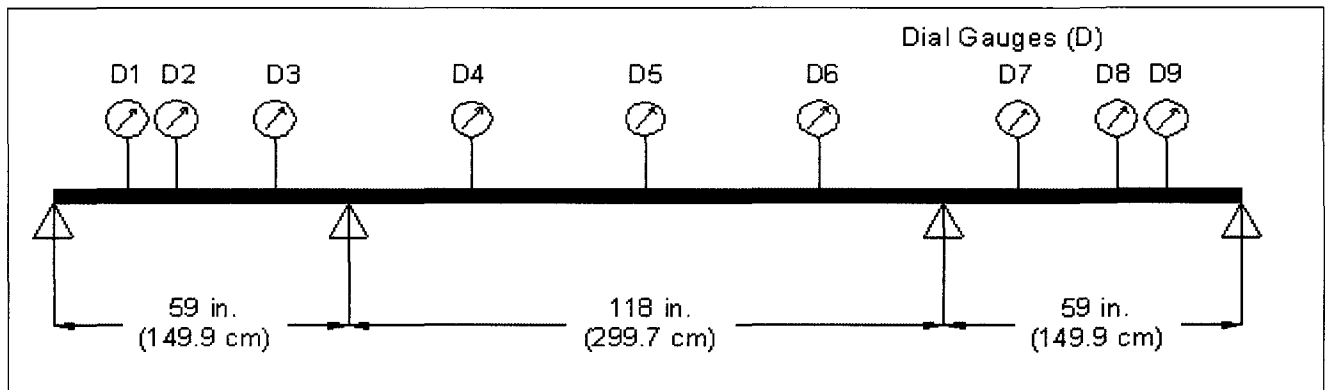


Figure 4-3: Dial Gauge/LVDT layout

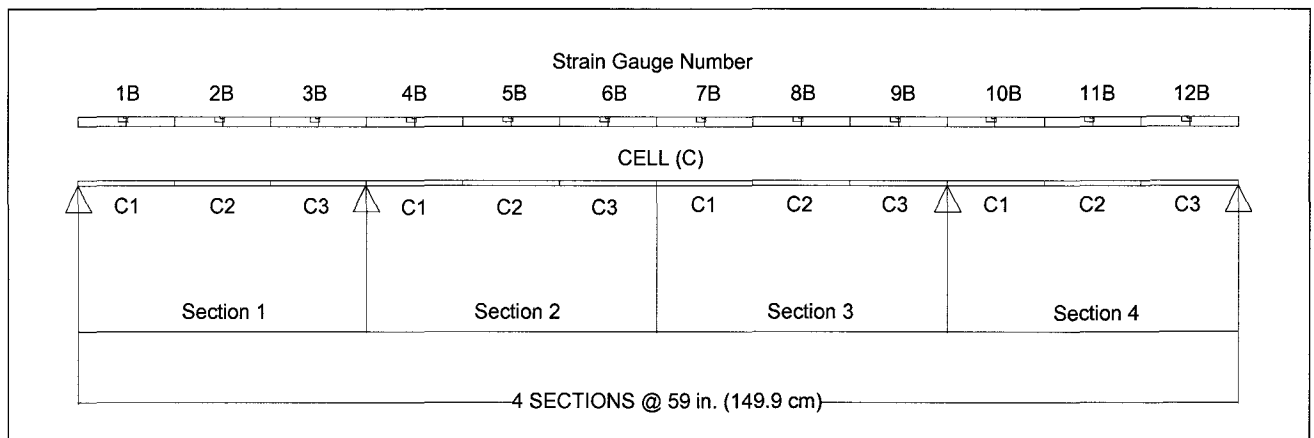


Figure 4-4: "Ideal 12 Strain gauge system layout"

A static load test was performed on this beam to compare the performance of the strain gauge system with the LVDTs. The beam was loaded with 18.71 lbs at the center of the center span. Figure 4-5 shows the deflected shape determined numerically from the Electronic resistance strain gauges compared with the LVDTs. The figure also shows the values that were derived from a SAP2000 model.

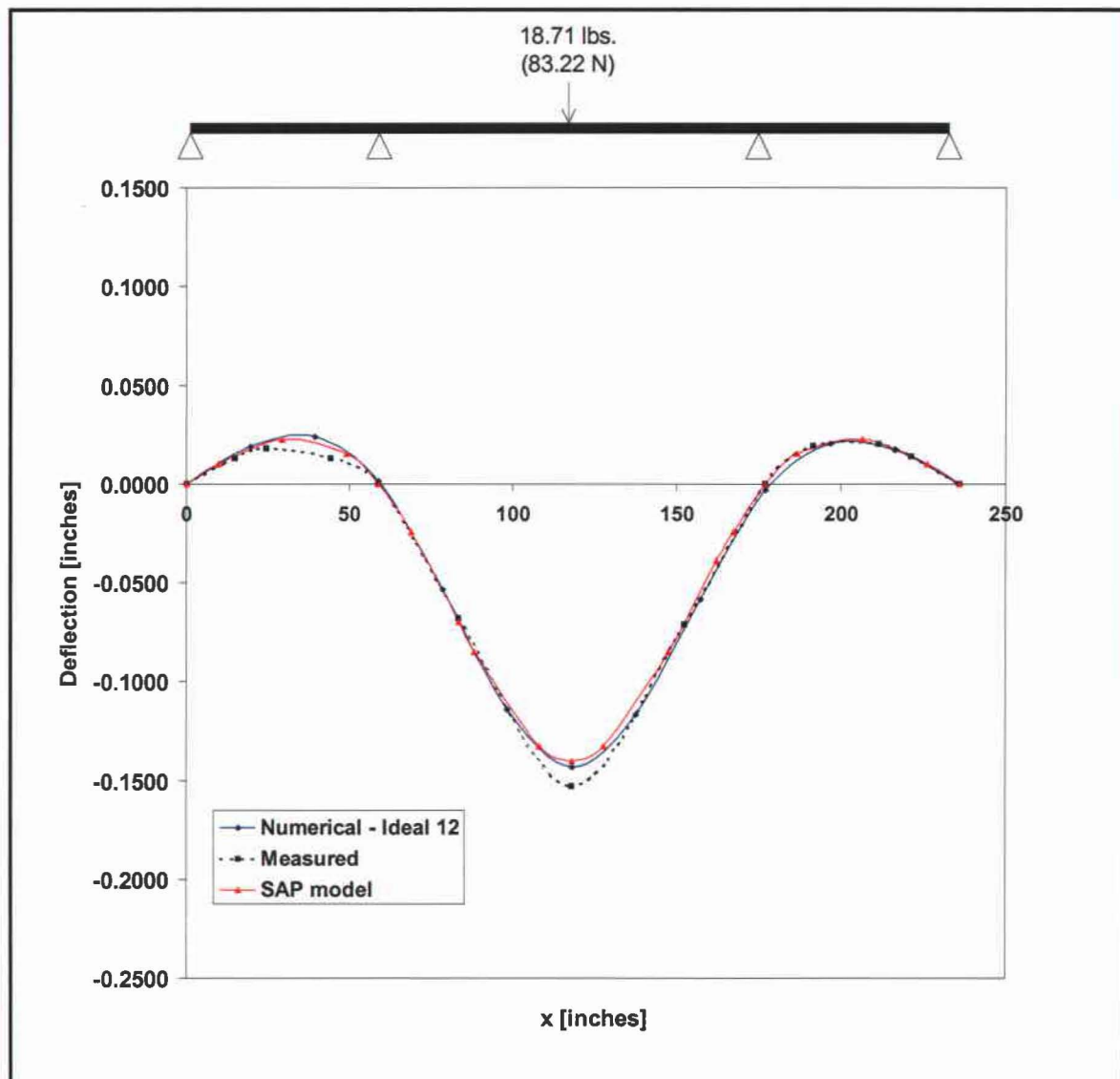


Figure 4-5: Comparison of Strain gauge data to LVDT readings and SAP2000 output

The data and the previous figures showed that the strain gauge data agreed fairly well with the data from the LVDTs and the SAP model, therefore the Strain gauges were to be tested under dynamic conditions.

4.1.2 Dynamic testing of Strain Gauges

Dynamic tests were performed on the beam using the “ideal 12” electrical strain gauge system. The mid-point of the center span was pulled down by weights attached to

the beam by a piece of string. The string was then cut and the beam was allowed to vibrate freely. After a few cycles, the free vibration approximated the fundamental mode shape for the beam. SAP2000 estimated the theoretical natural period of the first mode of vibration at 0.0684 seconds. Several dynamic tests were run on the beam using this procedure. The strain gauges were scanned sequentially by a high-speed data acquisition system at a frequency of 8200 Hz. Each gauge was therefore scanned approximately 680 times/second. Assuming a natural period of 0.0684 seconds, there were approximately 48 readings/cycle. It should be noted that the data-acquisition system used could not read the whole set of twelve strain gauges simultaneously. Therefore, there was a split second lag between each reading

When the first mode of vibration is induced, the maximum strain occurs in gauge 6B at the middle of the central span. The strain measured from Strain Gauge 6B is plotted versus time in Figure 4-7. After a few initial cycles, the free vibration becomes repetitive. One representative free vibration cycle of data was processed to produce 48 separate deflected shapes as shown in Figure 4-6. From the representative cycle of data, it was calculated that the natural period from the dynamic test data was 0.07 seconds.

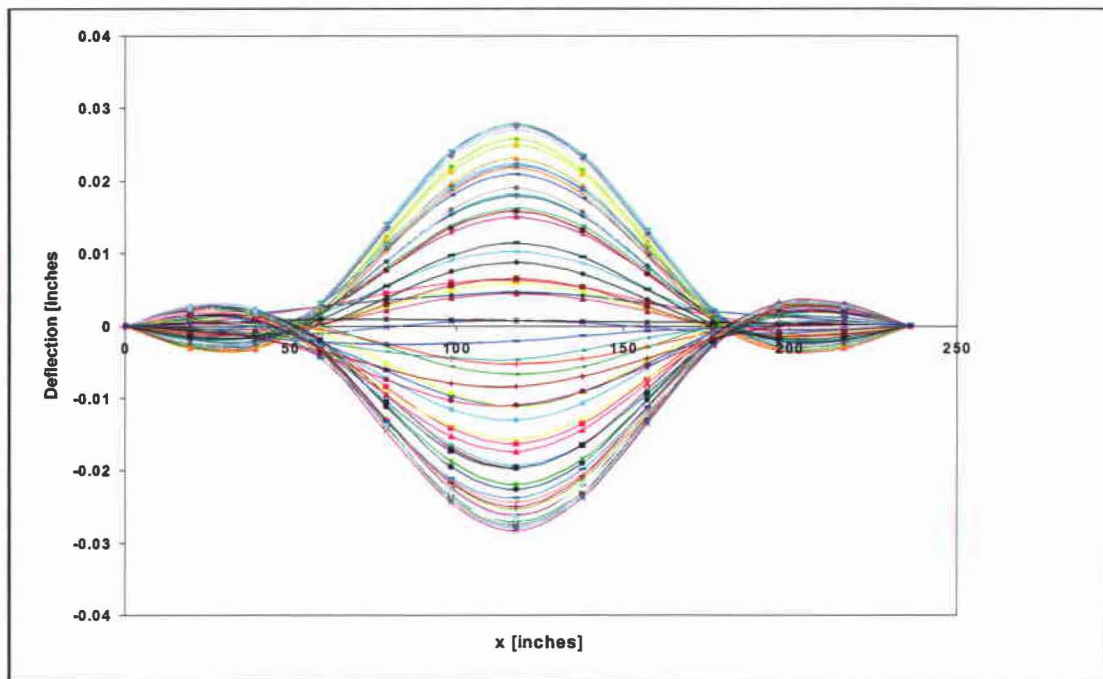


Figure 4-6: "Ideal 12" Dynamic test results

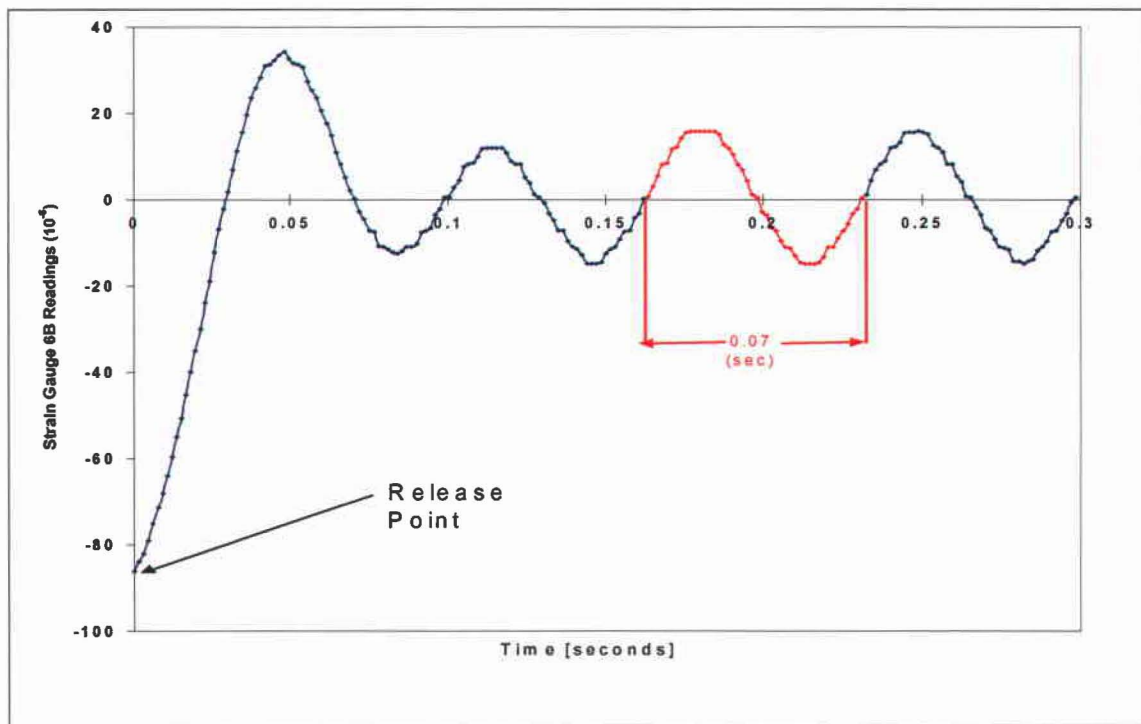


Figure 4-7: Vibration cycle of strains in Strain Gauge 6B

In order to compare the numerical deflection with the analytical mode shape obtained from SAP2000, the deformed shapes were normalized, assuming that the interior supports did not move. The natural period from the dynamic test data was 0.07 seconds. This agreed well with the natural period from the SAP2000 model of the beam, which was 0.0684 seconds. Note that the data acquisition equipment did not record the time steps to more than three digits of accuracy. Therefore, that could be a source of error for the period obtained.

The dynamic test performed on the tube induced the first mode shape of the beam. This mode shape was compared to the first mode shape obtained from SAP2000, scaled to match the peak mid-span deflection. The shapes were very similar (Figure 4-8).

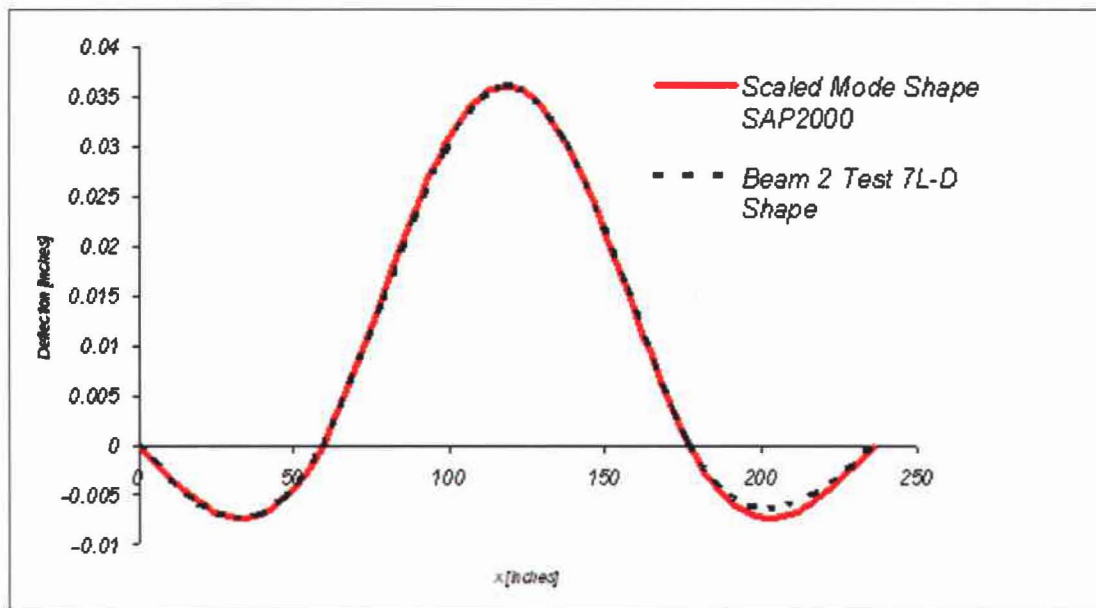


Figure 4-8: Mode Shape comparison

Although the deformed shapes in both Figures 4-9 and 4-10 look similar, the deflections calculated using the mathematical model are about $\frac{1}{4}^{\text{th}}$ the value of the “LVDT measured” deflections. It was also noted that there was a phase lag between the

strain gauge data and the mathematical model. From these observations, it was deduced that the weldable electrical resistance strain gauges are not capable of registering the correct strain experienced by the steel beam. This is because the substrate that the strain gauges are attached to cannot transfer the strains experienced in the beam to the gauge during high frequency vibration. The Vishay Measurements Group, Inc. was contacted and they explained that the weldable strain gauges are not as accurate as the epoxy-installed strain gauges for dynamic measurements. Their gauge factor could vary 5-10% from the specified value. Furthermore, the weldable strain gauges do not work well for dynamic loads. Also, these strain gauges experience a shear lag, meaning there will not be a full transfer of strain from the beam to the strain gauge. The supports had slight movement as shown in Figure 4-10, but this was not significant.

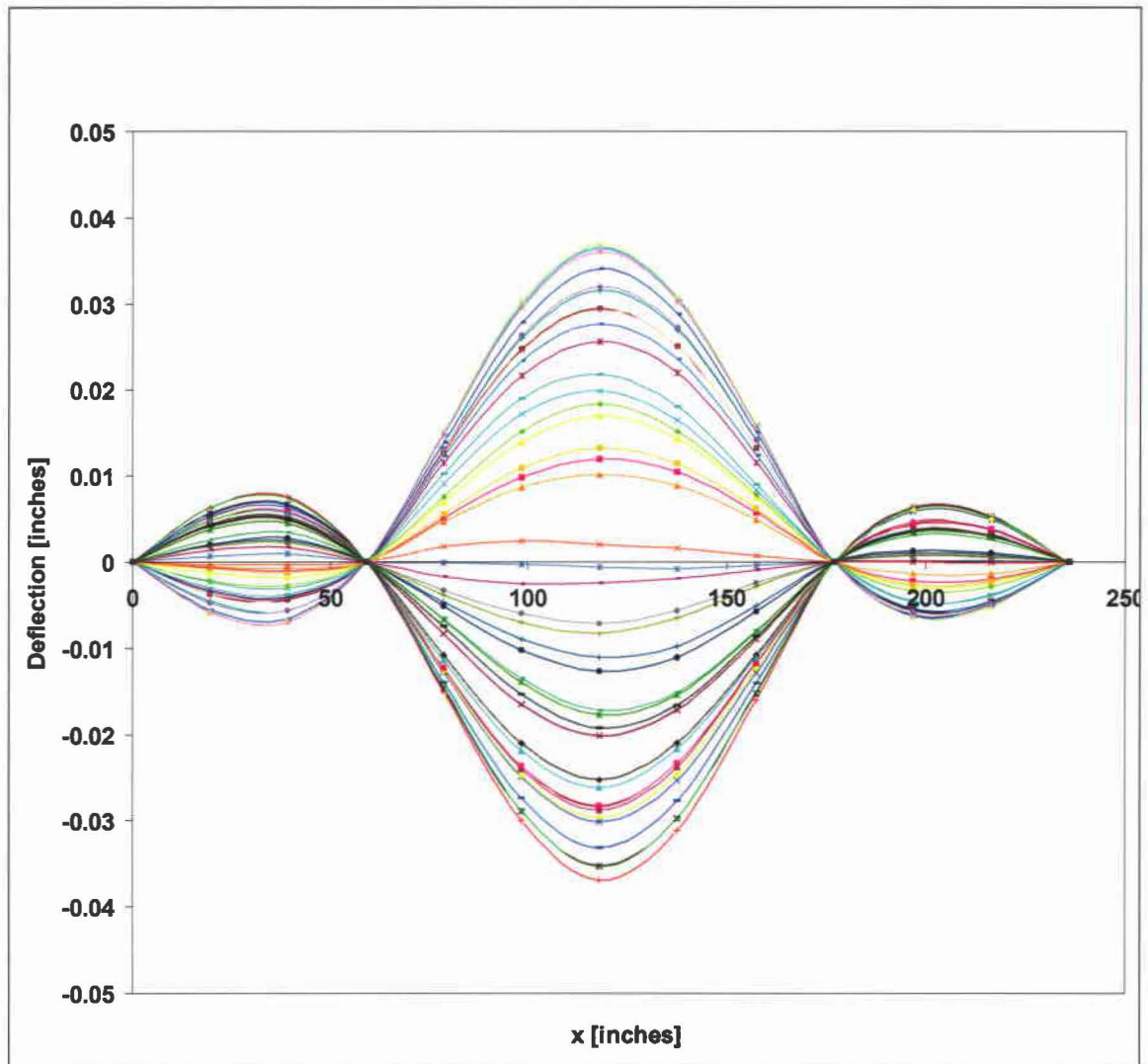


Figure 4-9: Normalized values of “Ideal 12” Strain gauges

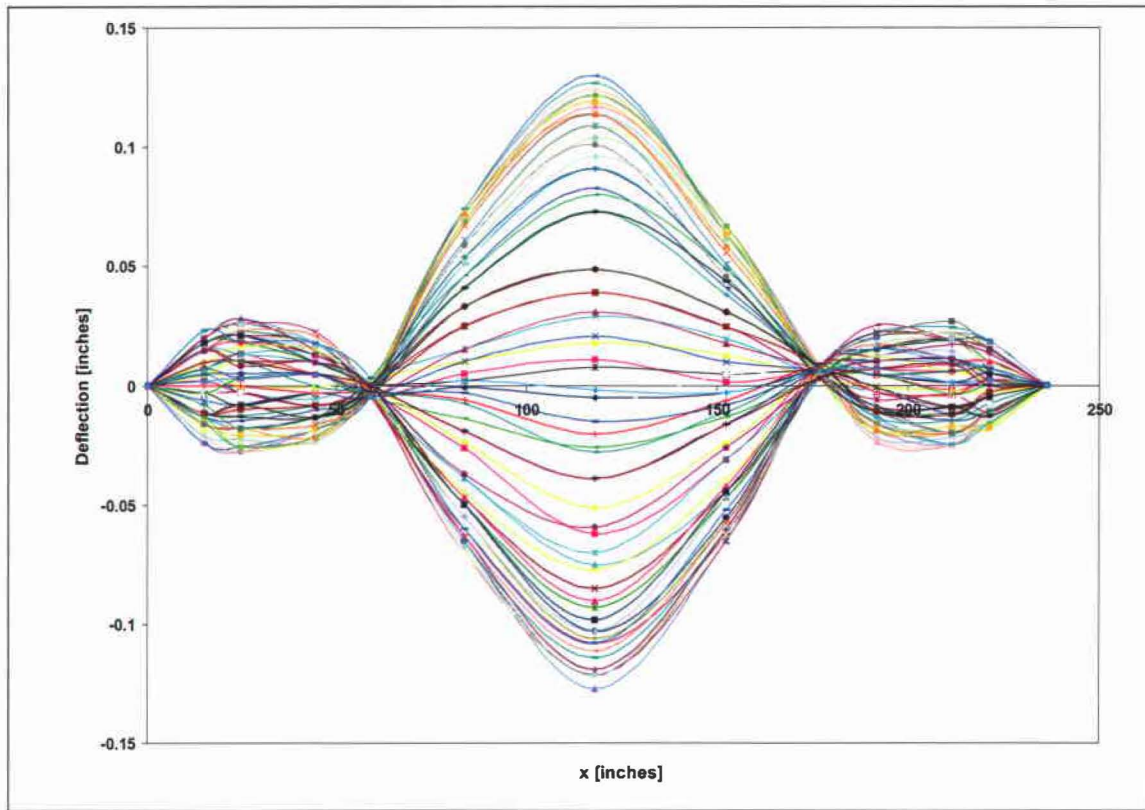


Figure 4-10: LVDT results for Dynamic test

The tests proved that the Electronic Resistance Strain Gauges (ERSG) could not provide an accurate display of the beam behavior in dynamic conditions. Therefore fiber Optic strain gauges were investigated to see if they could provide reliable data for the beams movement in a high frequency event.

4.2 Verification of Fiber Optic Strain Gauges

4.2.1 FO Sensors

The Fiber Optic gauges used were Fiso technologies Fabry-Pérot 5.0 mm Non-Compensated strain gauges with a range of $\pm 1000 \mu\epsilon$. A Fabry-Pérot gauge is based on a Fabry-Pérot interferometer (FPI), which consists of two mirrors facing each other, the space separating the mirrors is called the cavity length. Light reflected in the gauge is

wavelength-encoded in exact accordance with the cavity length. When bonded to a specimen the strain transferred to the gauge is converted into cavity length variations and then, in engineering units by a signal conditioner.

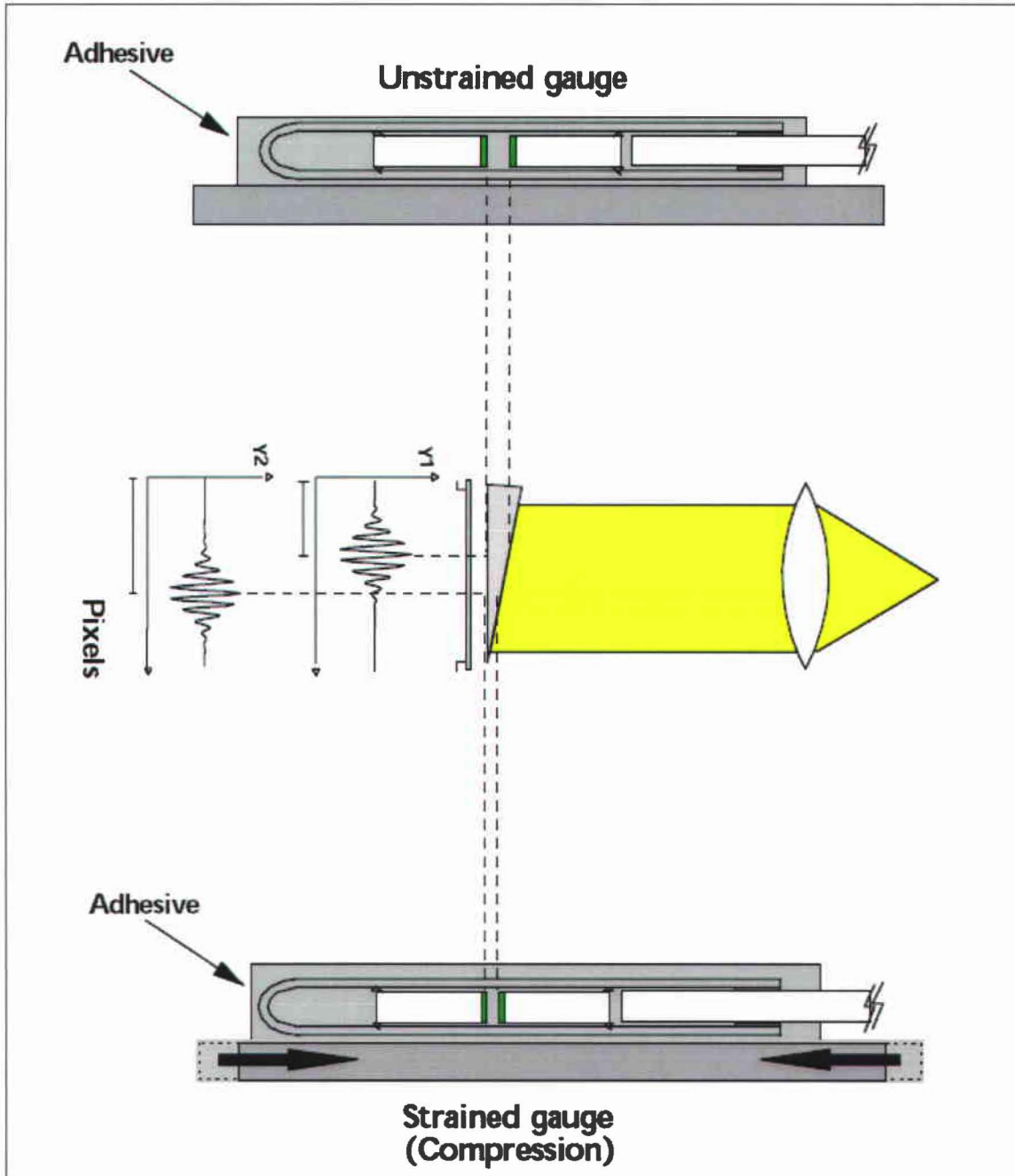


Figure 4-11: Fiso Fabry-Pérot gauge operation [Fiso Technologies Inc., 1997]

The Non-Compensated gauge is one that does not account for the effects of thermal expansion. In a N.C. fiber optic strain gauge the gauge senses a strain level equal to $\Delta T \cdot \alpha$, where α is the coefficient of thermal expansion of the specimen. The coefficient of thermal expansion of the fibers can be as low as $0.5 \mu\epsilon/^\circ\text{C}$; therefore the gauge measures the expansion of the material whether the expansion is due to mechanical stress or thermal dilatation.

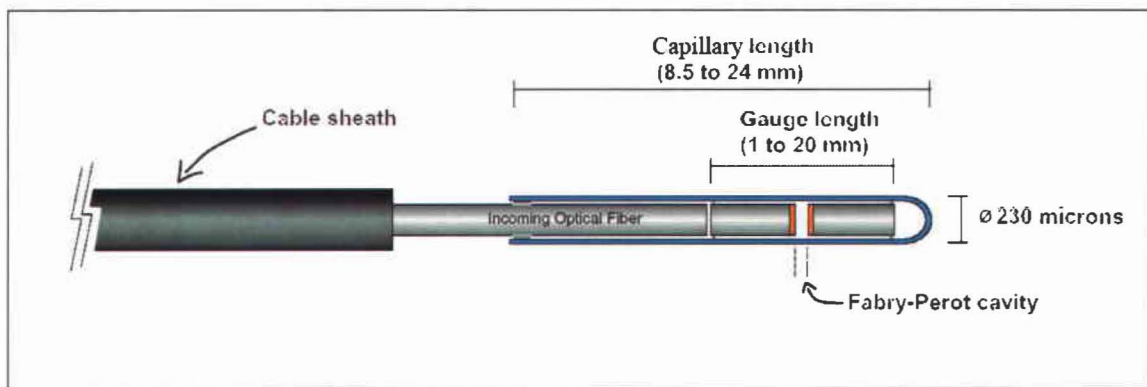


Figure 4-12: Fabry-Pérot strain gauge

In order to attach the gauges to the steel tube each gauge location was sanded down with 360-grit sandpaper. The process of cleaning and grinding each location is similar to the process used to attach the Electronic Resistance Strain gauges. The gauges were then coated with an adhesive and attached to the tube. The wires attached to the gauge were taped down and epoxied to the beam.

Symmetry was assumed and six gauges were installed in the same pattern as the electronic resistance strain gauges, one gauge at the center of each cell of one-half of the beam. The fiber optic pattern also used an additional six gauges on the bottom of the beam, to test for the effects of torsion. The fiber optic gauges were installed opposite of

the electronic resistance gauges on the same face as the beam. The cross section can be seen in figure 4-2.

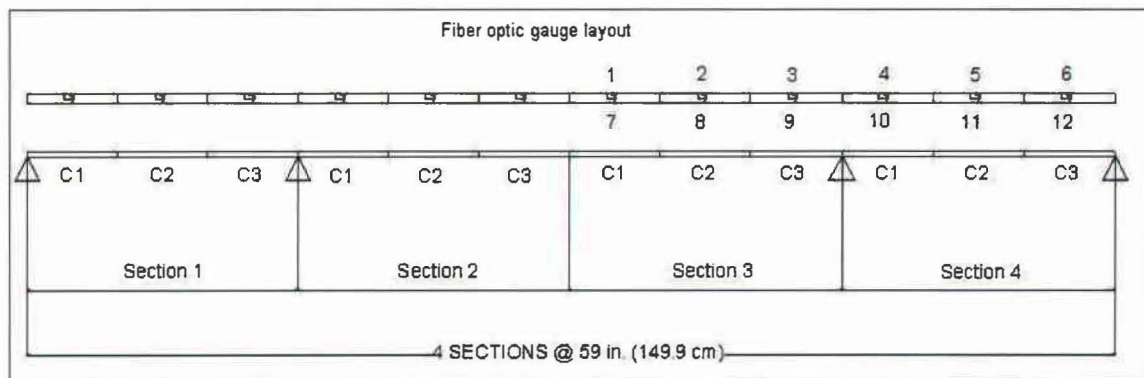


Figure 4-13: Fiber optic gauge layout

The fiber optic strain gauges were attached to a 16 card BUS system chassis. However, only 12 strain gauges were purchased so only 12 of the channels were used. A company called RocTest Inc., in collaboration with Telemac S.A., and another company called Sensoptic created the data acquisition software provided by Fiso technologies. However, the data acquisition card used to collect the data was made by National Instruments. Therefore, LabVIEW was used to collect and store the data.

4.2.2 *Dynamic test of strain gauges*

In order to test the values being output by the Fiber optic strain gauges a simple static step-loading test was run. The programs for the ER gauges and the FO gauges were started at the same time then weight was added to the beam. The weights were added in two-pound increments to 8 lbs, and then the load was taken off again at the same intervals. The outcome of the test can be seen in Figure 4-14. The figure shows that the micro strain values for the FO gauges come out fairly close to the values for the ER

gauges, therefore the FO gauges were to be tested under dynamic loading. At a peak strain of $31 \mu\epsilon$ the difference between ER and FO strain gauges was $2 \mu\epsilon$ or 6.5%.

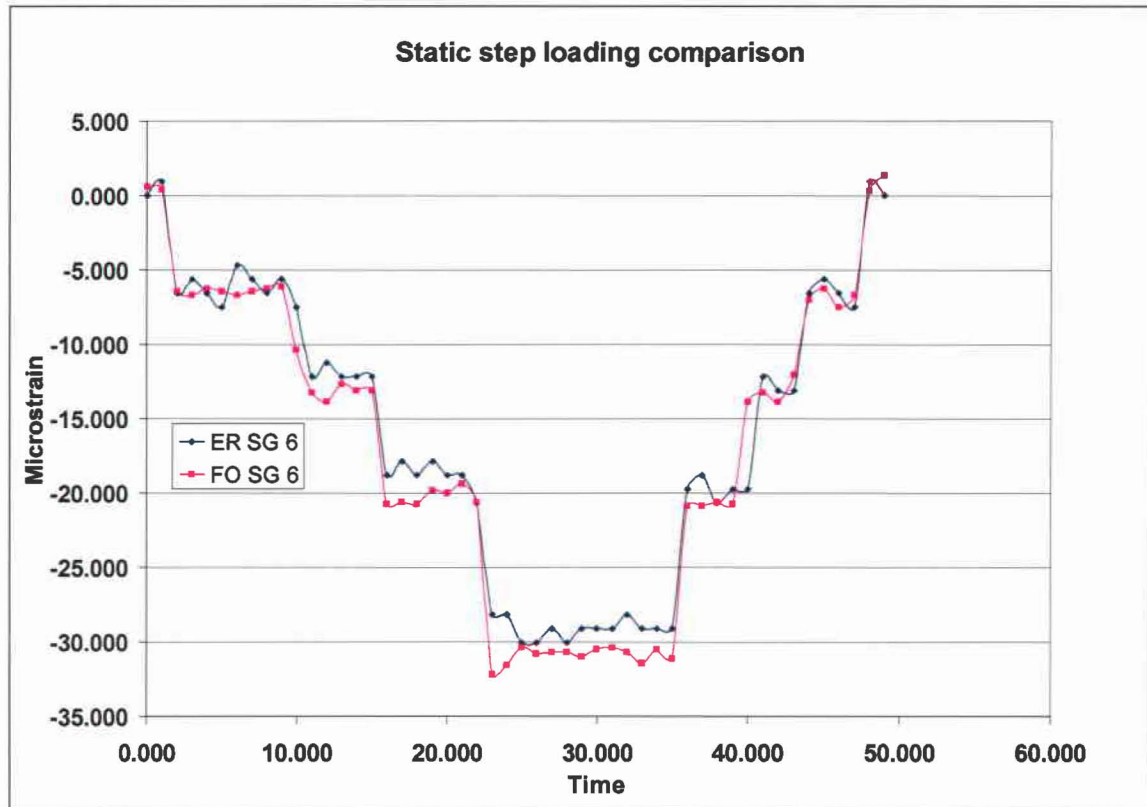


Figure 4-14: Static step loading comparison of FO and ER gauges

The set up for the FO strain gauge dynamic test was similar to the ER strain gauge dynamic test. A string was again attached to the beam and connected to weights. After the string was cut the beam was allowed to vibrate and the movement was recorded. The deflections in the Fiber Optic gauges showed the beam to be moving at the supports again, the non-normalized values for the deflections can be seen in Figure 4-15. The normalized values at the supports and the outcome can be seen in Figure 4-16. The values for the LVDTs are in Figure 4-17.

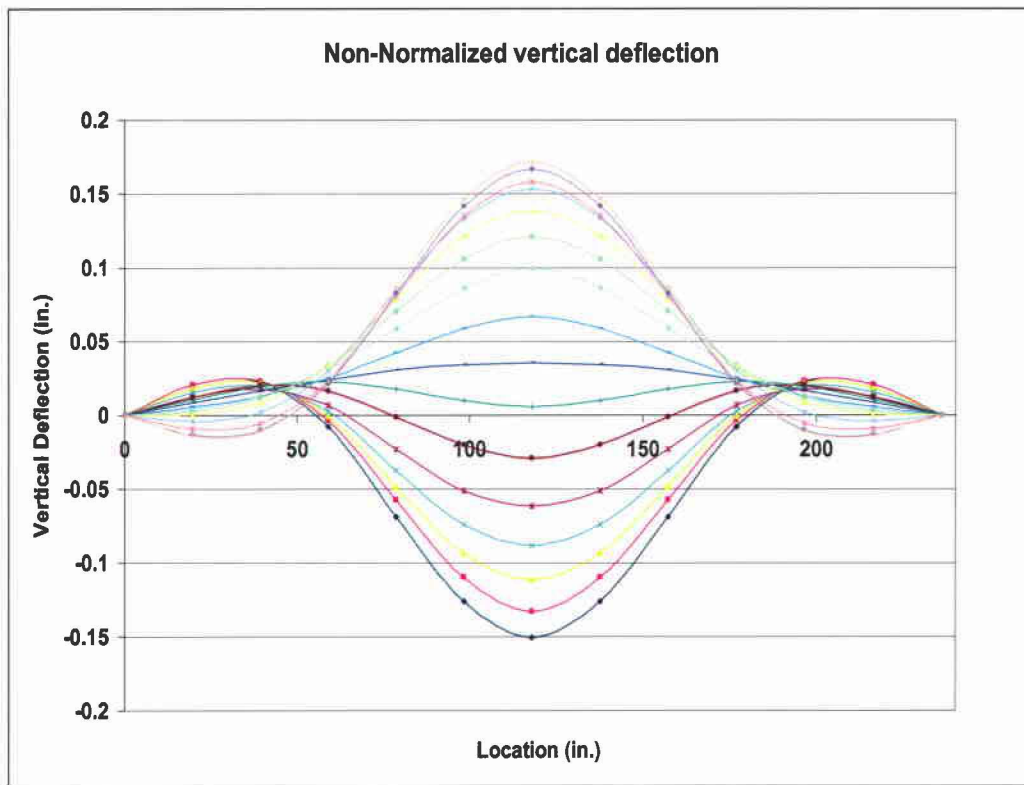


Figure 4-15: Non-Normalized vertical deflection

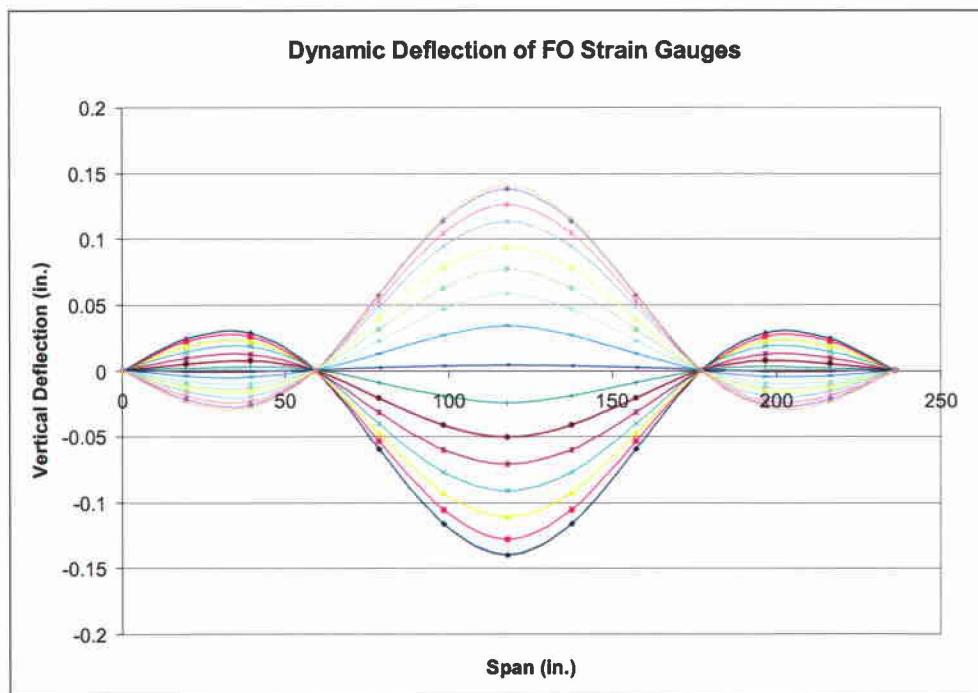


Figure 4-16: FO strain gauge dynamic deflections

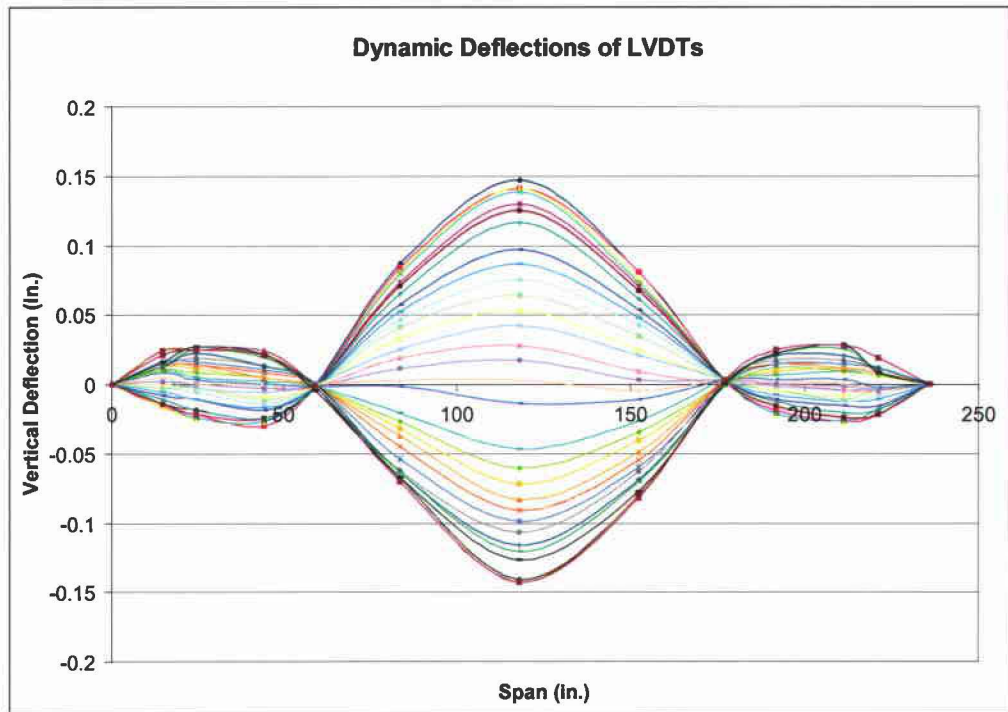


Figure 4-17: LVDT dynamic deflections

Figure 4-18 shows that the FO strain gauges performed very well under the dynamic conditions. The figure shows the normalized Fiber Optic strain gauge deflections inside of an LVDT envelope, it shows a very good agreement between the two systems. This information was used to verify that the FO gauges could be used in a high seismic activity.

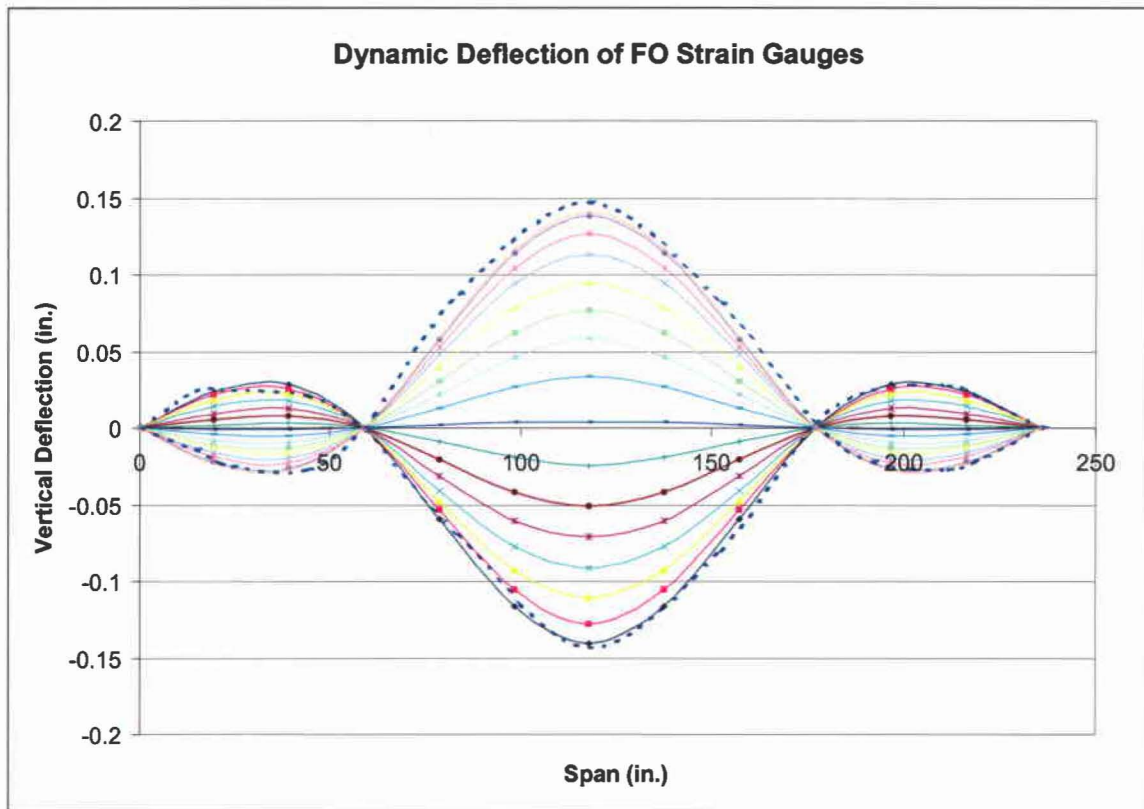


Figure 4-18: Normalized FO deflections with an LVDT envelope

4.2.3 Torsional testing of strain gauge system

One additional test run on the beam was a torsional test. The test was performed to determine if the strain gauge deflection system could be used to capture all the effects on the beam, such as axial stress, bending, and torsion. On the half span of the beam with Fiber Optic strain gauges on the top and bottom, six more Electronic resistance strain gauges were added to the bottom. The gauges were added in the same location on the opposite face of the beam. Again, the placements of these gauges are shown on Figure 4-1.

The test consisted of two fish scales and the weights previously used to conduct the static experiments on the beam. A $\frac{1}{4} \times 1 \frac{1}{2} \times 12$ " steel plate was also used to create

the torsion effect. To create the effect the beam was turned on its side and the steel plate was attached long ways with a vice grip to the mid-span. A string was also tied around the beam at the same location. The string was used to hang weights and represent the effects of bending in the x-direction (F1). One fish scale was attached to the mid-span and to a block perpendicular to the beam; this scale was used to represent bending in the y-direction (F2). The last scale was used for the torsion effect; it was attached to the end of the steel plate and to the same block (F3). The forces and their respective locations are shown in Figure 4-19. The test was also conducted with F3 acting in the opposite direction.

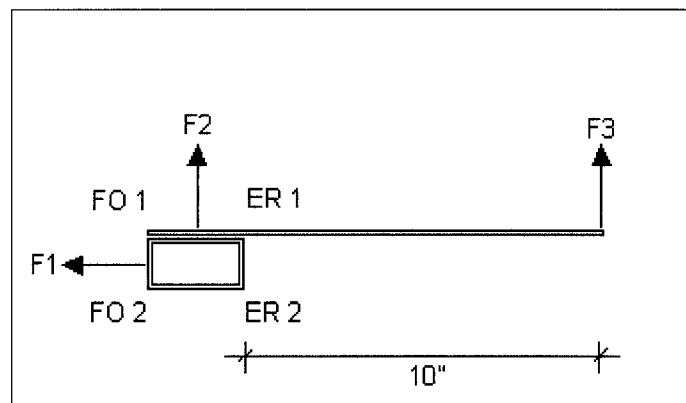


Figure 4-19: Loads on steel tube beam

After completing the torsion tests and compiling the results it became clear that this experiment could not yield the results we were looking for. Figure 4-20 shows the deflection in mid-span as the beam is subjected to just the F3 loading as shown in the figure.

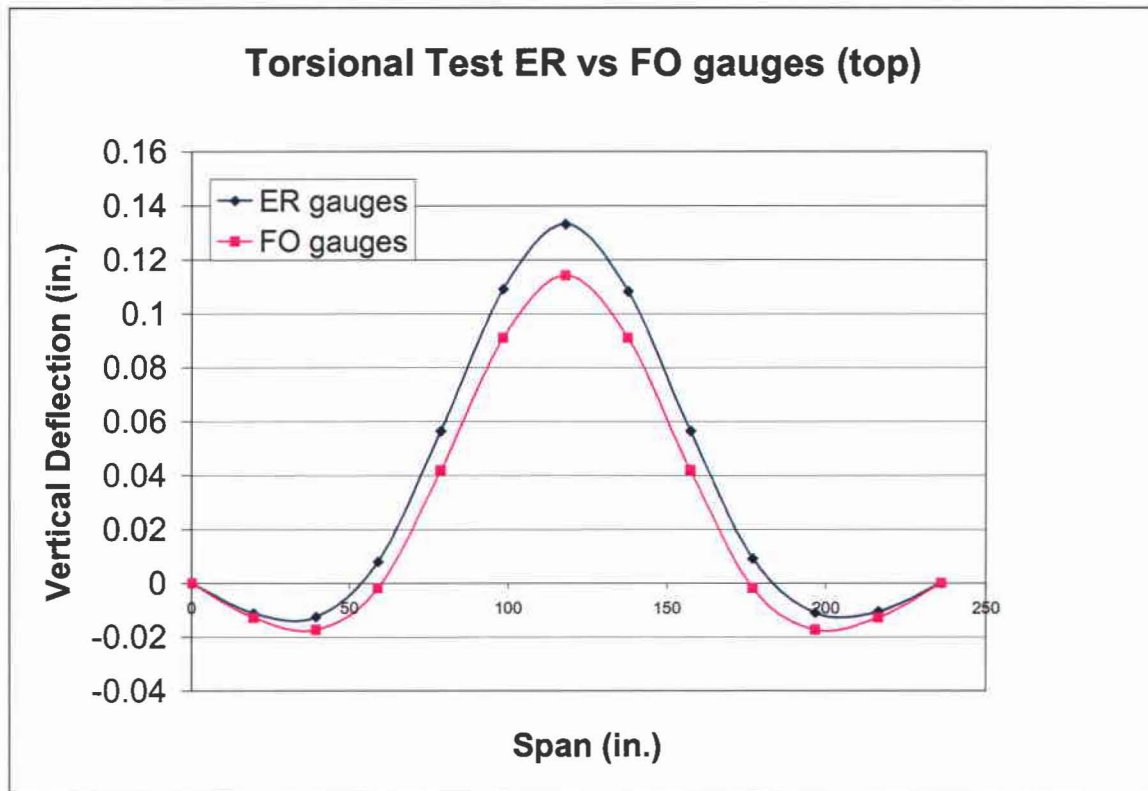


Figure 4-20: Deflection comparison of gauges due to torsion

After applying the load in this manner it was expected that the ER gauges would be slightly higher at the mid-span than the FO gauges. At first glance it looks as if the data was entered improperly, but numerous tests confirmed that the phenomenon occurring is what the gauges are reading. An example of this is seen in Figure 4-21. The figure shows the vertical deflection of the beam as it's subjected to F3 in the opposite direction, a clockwise rotation, according to Figure 4-19. That test also gave values that were opposite of what was anticipated.

The discrepancy in the output could be due to the fact that gauges can only read the axial strain in the beam. The problem with this is that there is axial strain in the beam that is a result of bending and twisting. The strain that is being read by the gauge cannot

determine from which action the strain is coming from. The alternative to this experiment will be discussed in the recommendations section.

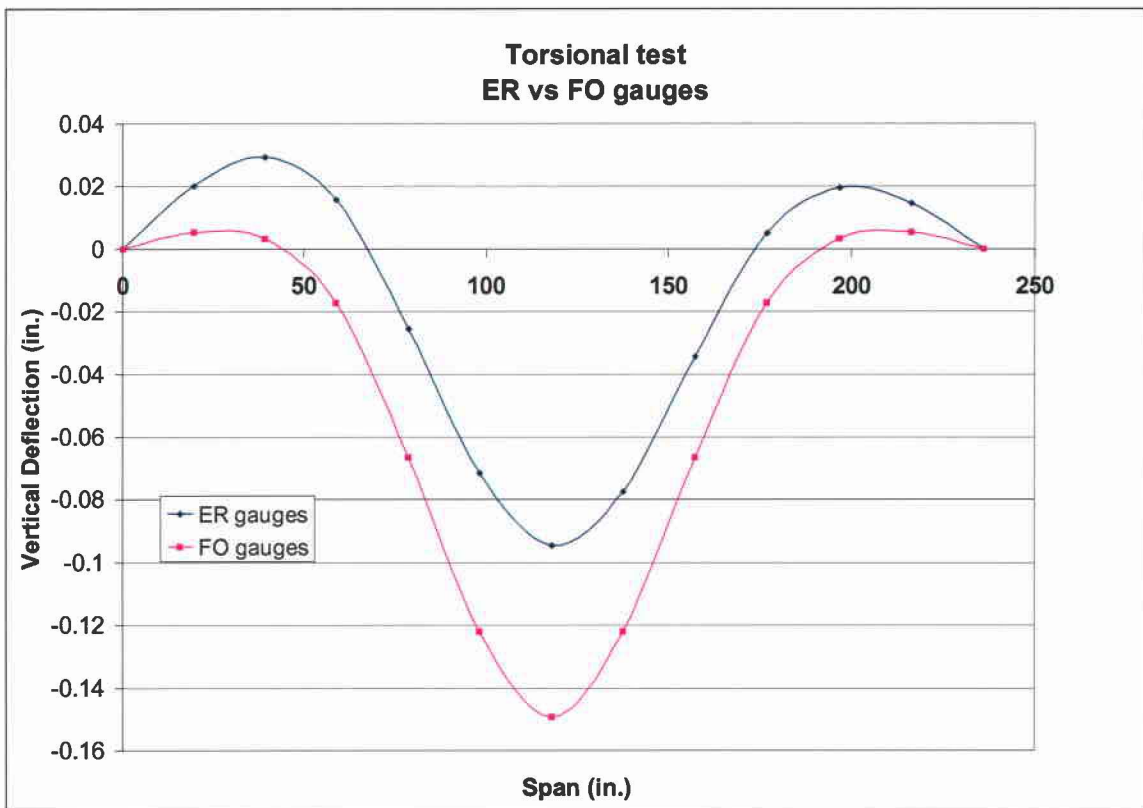


Figure 4-21: Deflection of gauges due to torsion in the opposite direction

4.3 Strain gauge system applied to the North Halawa Valley Viaduct (NHVV)

After validating the strain gauge system on the Laboratory beam under static and low frequency deflections a similar strain gauge system was installed in the NHVV along side the Base-line LVDT system described earlier. This system was installed in the bridge span between piers P8 and P9. The span was divided into six equal cells, with a strain gauge located at the center of each cell. The gauges were placed along the centerline of the bridge at the top and bottom of the girder at each position.

In order to measure the average strain in the concrete, a longer gauge length was required than the 2 inch gauge used in the laboratory test. To achieve the same results as the laboratory model it was determined that the gauges should be approximately one meter in length. A one-meter gauge length strain gauge was fabricated in the laboratory. Weldable electric resistance strain gauges were affixed to a meter long steel tube (Figure 4-22). Two $\frac{1}{4}$ in thick plates were welded to either end of the tube. The reading from the gauge would represent the average strain over a meter of the box girder. In order to attach the gauges to the box girder, four $\frac{3}{8}$ in diameter holes were drilled in each plate for wedge anchor bolts.

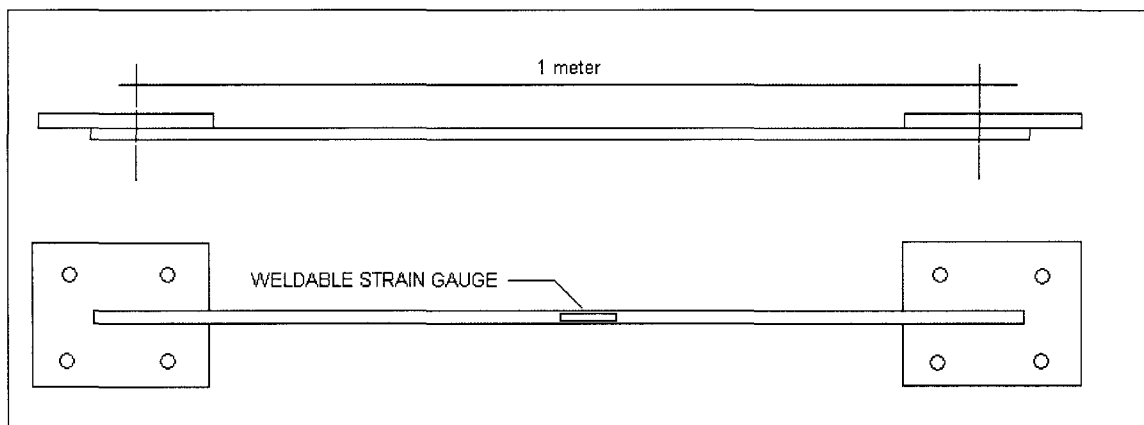


Figure 4-22: Fabricated meter long strain gauge

The strain gage data were recorded using the LabVIEW Data acquisition system. A computer and a conditioner card chassis were installed inside the box girder along with the meter long strain gauges and the LVDTs. The data were collected at the same time as the LVDT data. Figure 4-23 shows the strain values from the six strain gages installed on the bottom slab of the NHVV.

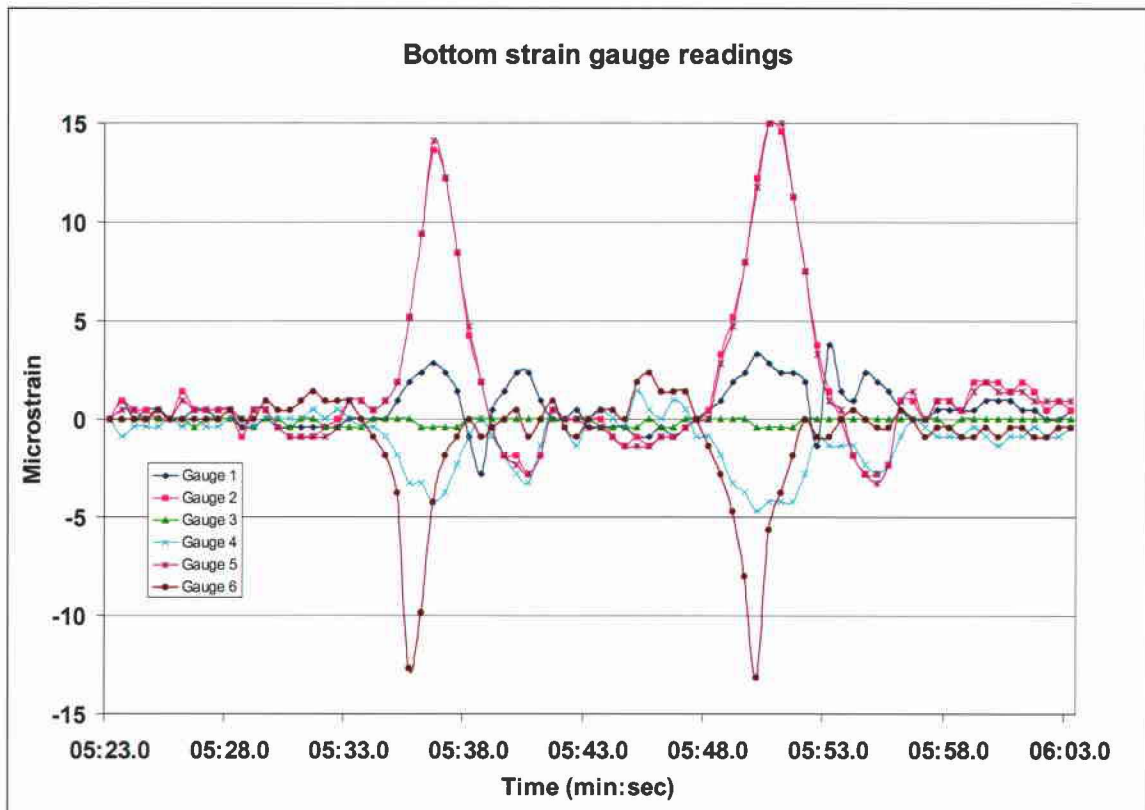


Figure 4-23: Bottom strain gauge readings from the NHVV

The time interval for the strain gauge data shown in Figure 4-23 is the same as used for the deflection recorded by the LVDT Baseline system in Figure 3-16. The strain gauges show two spikes at approximately 05:35 and at 05:51. These correspond to the deflection spikes recorded by the LVDT base-line system for two vehicles crossing the instrumented span. Apart from gauge 3 which recorded very little change in strain, possibly because of a gauge or data channel malfunction, the other gauges recorded significant strain values under the passing vehicle loads.

The data collected from the gauges were entered into the excel program used in the laboratory beam experiments. However, since the end conditions for the span P8-P9 are unknown due to continuity with adjacent spans, the program assumption of pinned end conditions was incorrect. As a result, the strain gauge system was not able to provide

the deflected shape for this interior span. In order for this system to provide bridge deflections, it is necessary to instrument the full bridge from pin support to pin support. Alternatively, rotations monitored at the supports could be used to modify the end conditions during deflection processing. Recommendations for future development and implementation of this system are discussed in the next chapter.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Modified Baseline systems

The modified Baseline system using LVDTs for automated deflection monitoring worked well in Laboratory trials, agreeing with the measured deflections during experiments on a concrete T-beam. The Base-line system was able to monitor the deflected shape of the beam even after the beam had cracked.

The baseline system also provided reliable data for vertical deflection of the box girder bridge in the North Halawa Valley Viaduct. The deflection of the beam due to diurnal temperature effects compared well with the expected values based on a previous thermal study of the NHVV. The baseline system was also able to record the vertical deflection of the box girder during vehicular traffic over the span. Although the actual displacement of the girder could not be verified due to lack of information on the weight of the passing vehicles, the behavior of the beam corresponds very well to the motion of a vehicle passing across the span.

For future applications of the LVDT baseline system, it is recommended that a heavy gauge piano wire be used (at least a #20 wire) with at least 200 lbs deadweight at the live end. It is also recommended that an improved pulley system be developed to minimize the friction at the live end and that the system be calibrated using a vehicle of known weight.

5.2 Modified Strain Gauge system

The strain gauge deflection system with fiber optic (FO) strain gauges showed excellent results in the laboratory dynamic tests on a three span steel tube beam. The deflected shapes derived from the FO strain gauges agreed well with the results of the Linear Variable Displacement Transducers. However, attempts to utilize this system to monitor torsion of the tube section were unsuccessful because the strains induced in the gauges are not exclusively from bending of the beam.

Application of this system to a Prestressed Concrete T-beam provided accurate deflection measurements until flexural cracking of the concrete. The 2 inch surface mounted electrical resistance strain gages bonded to the concrete surface were not able to monitor average strain after concrete cracking.

The meter long gauges installed in the NHVV field trial were able to record strains during traffic flow that compared well with the deflections recorded by the LVDT baseline system. However, the excel program developed to process the strain readings to determine the deflected shape could not be applied to these data because of continuity of the box girder over the end supports.

For future applications of this system to bridge deflection monitoring, it will be necessary to install strain gauges along the full length of the bridge unit from pin support to pin support. Alternatively, the rotation at continuous end supports could be monitored and used to update the end conditions assumed by the program.

In order to avoid additional strains induced by torsion in the box girder, it is recommended that the FO strain gauges be located along the centroidal axes of the cross-

section. For vertical deflection monitoring, gauges should be located along the centerline of the box girder (S.G. 1 and 3 in Figure 5-1), while for transverse deflection, gauges should be located along the horizontal centroid of the section (S.G. 2 and 4 in Figure 5-1).

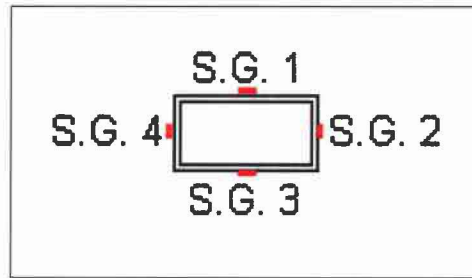
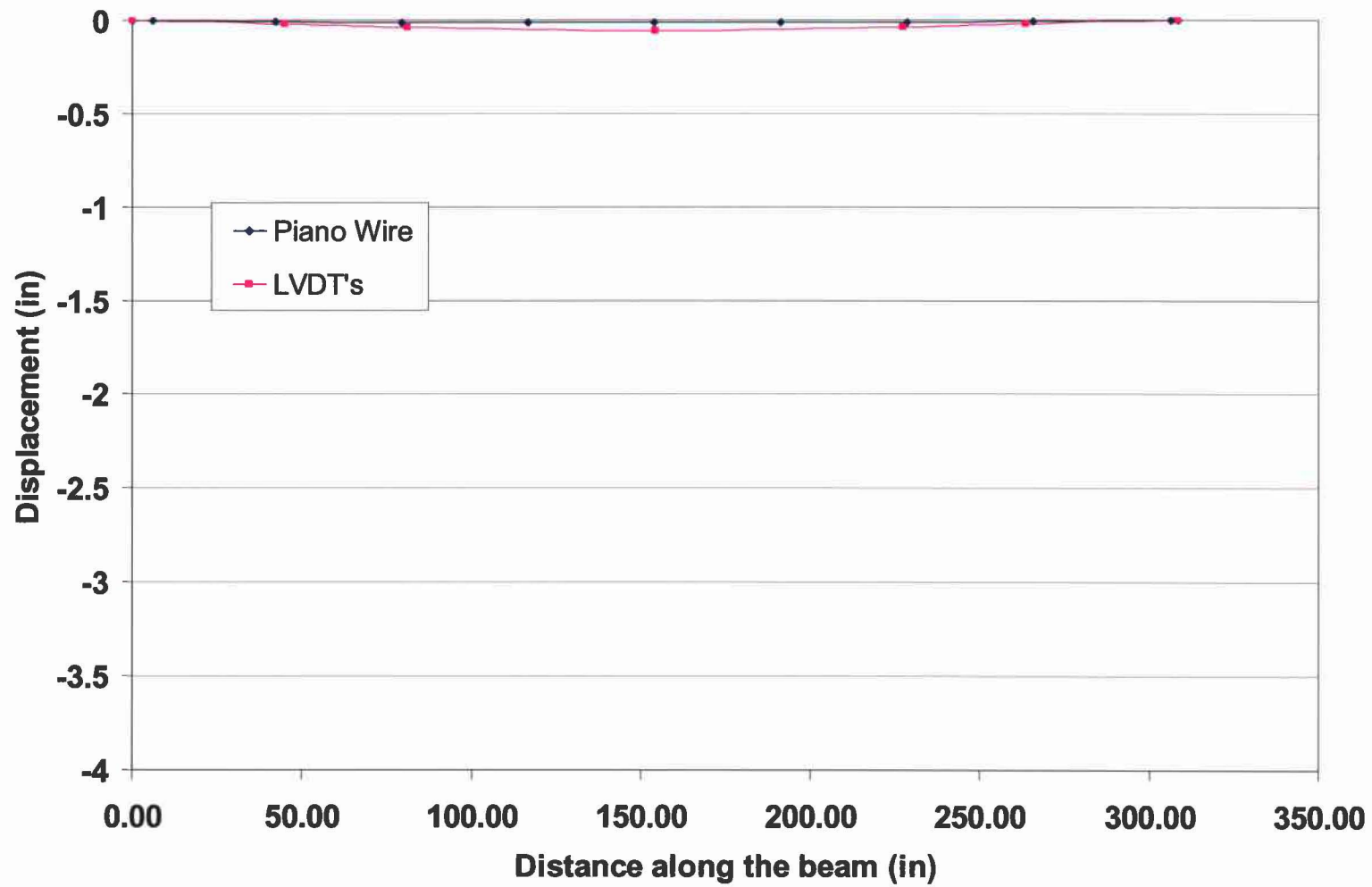
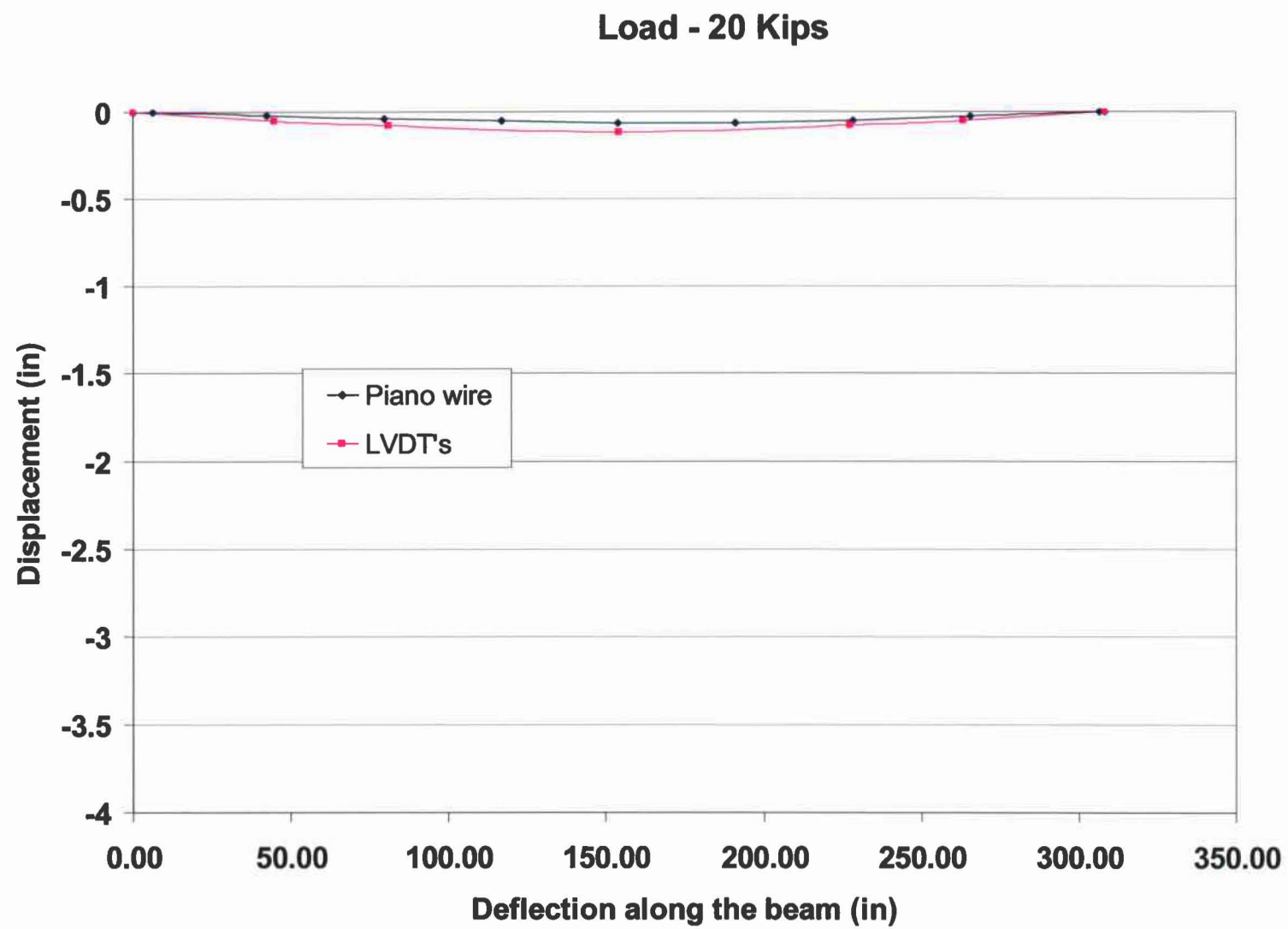


Figure 5-1: Recommended strain gauge locations

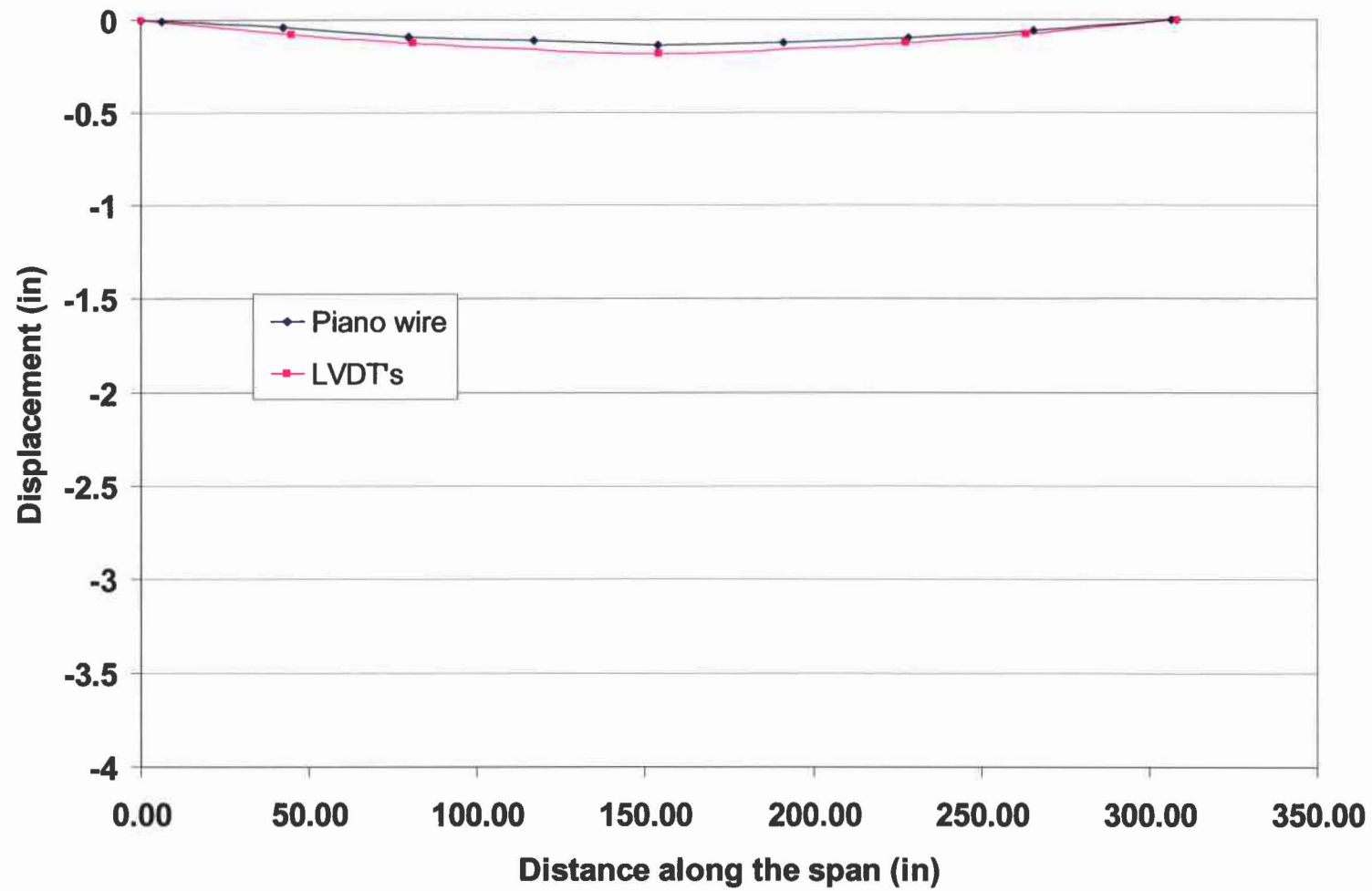
Appendix A: Base-Line vs. Measured Deflection Measurements

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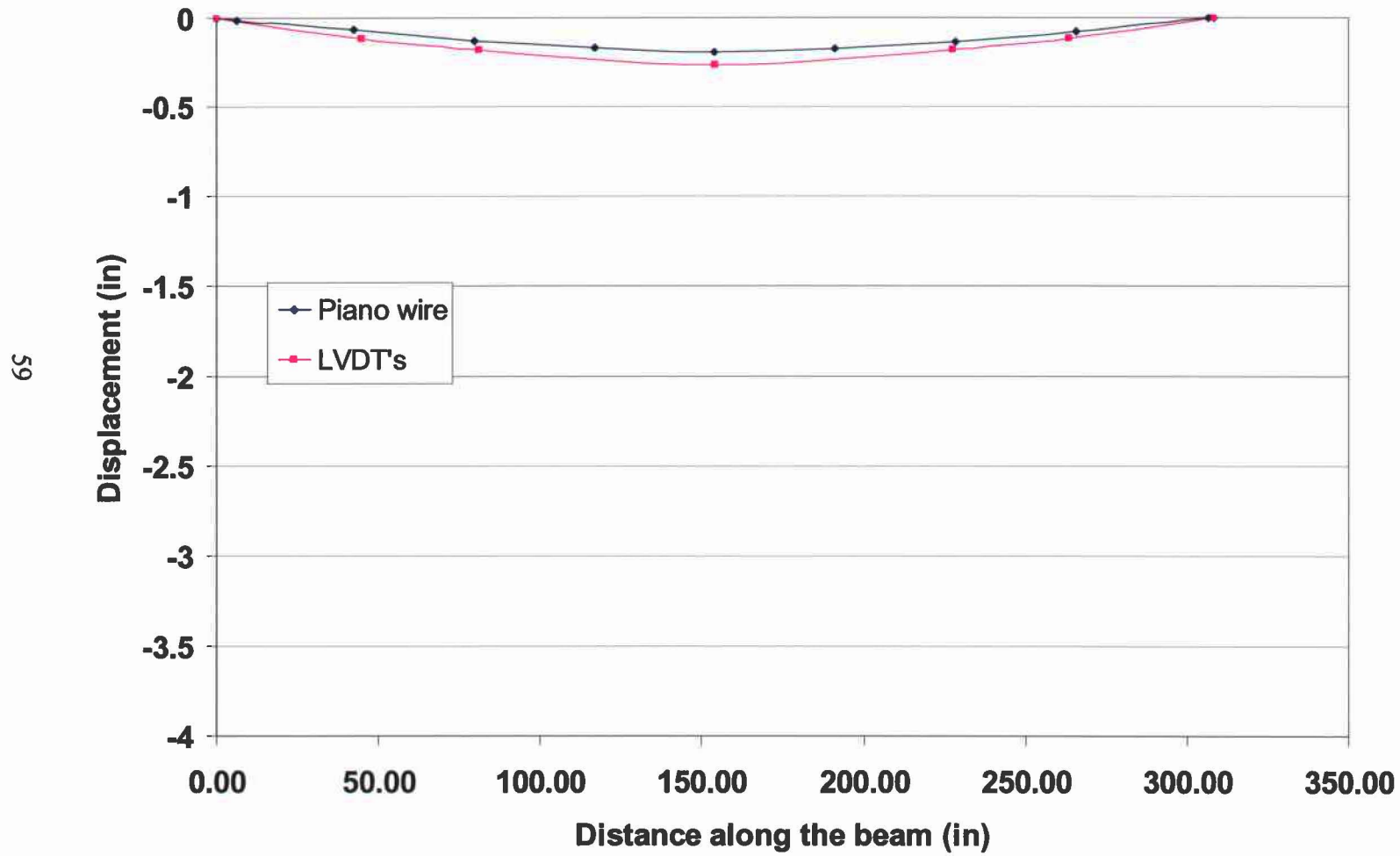




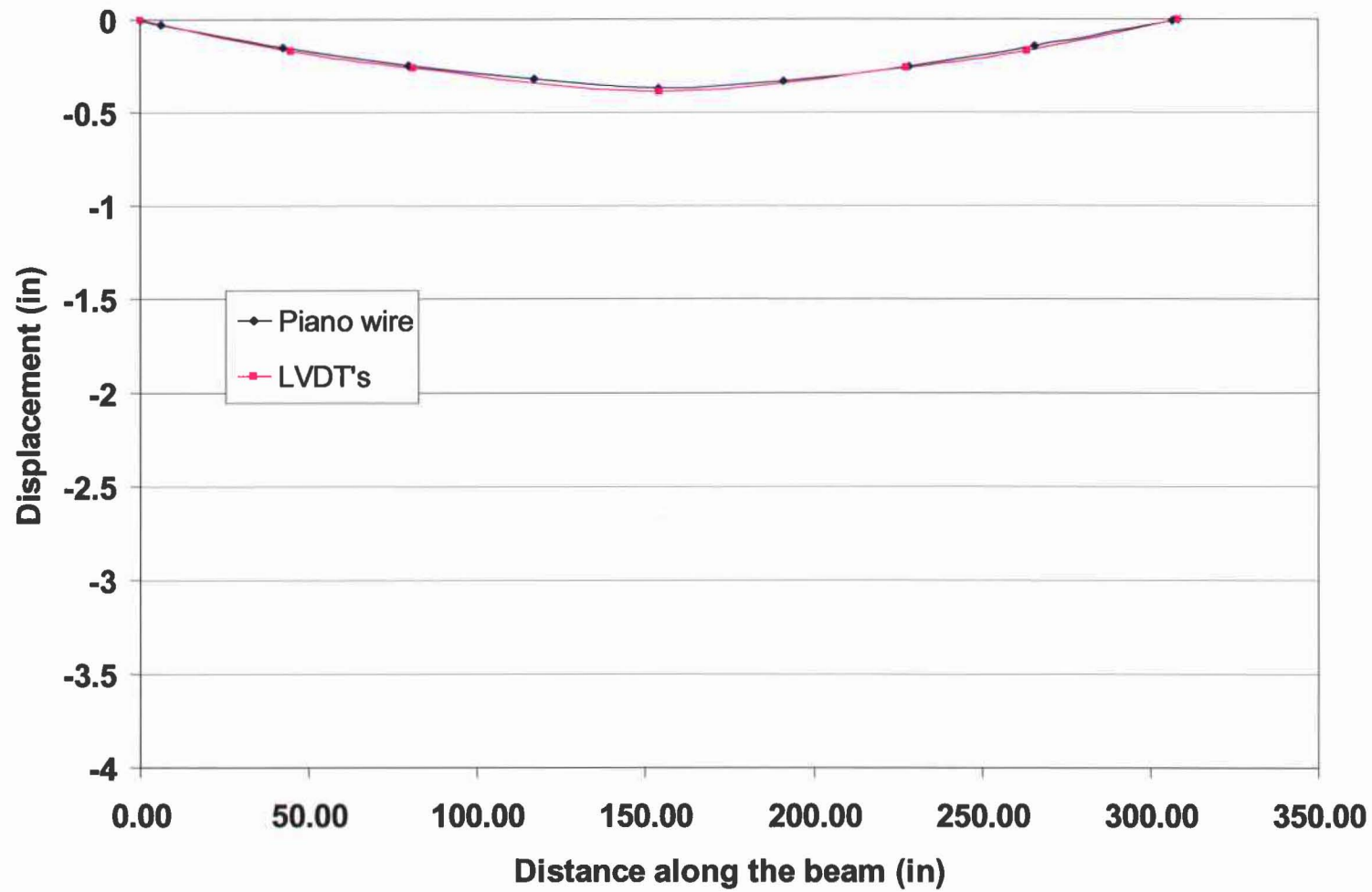
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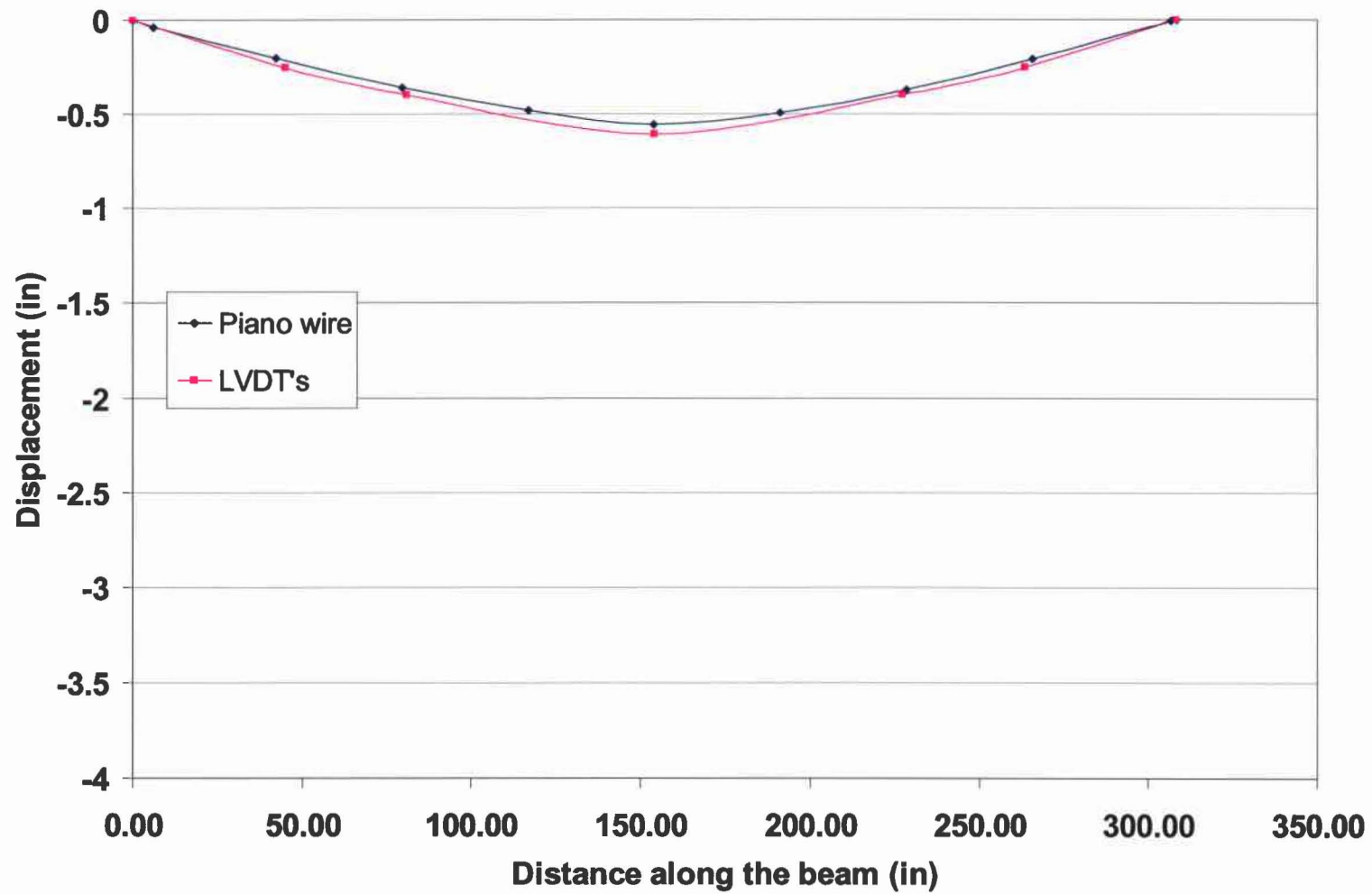
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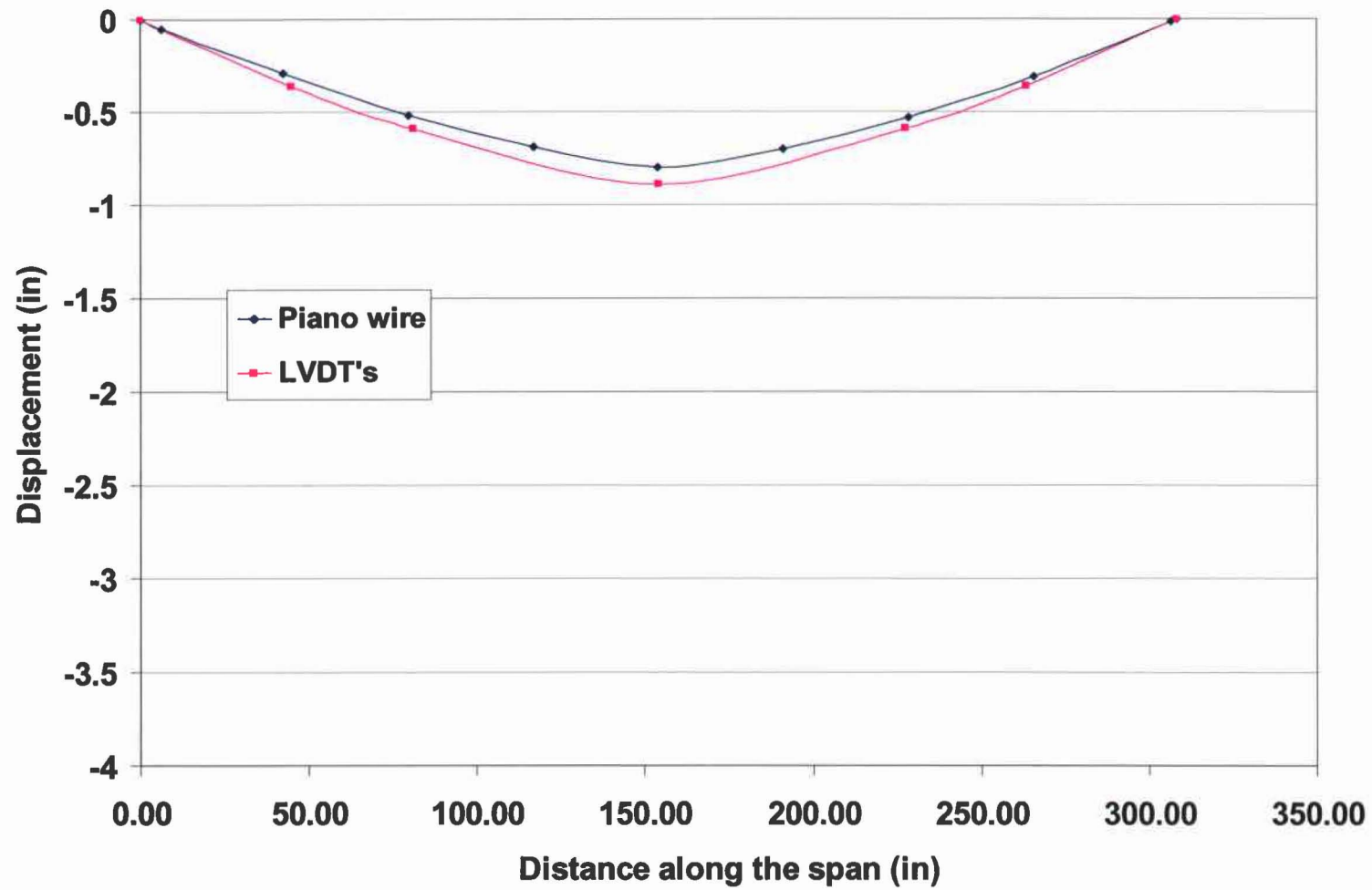
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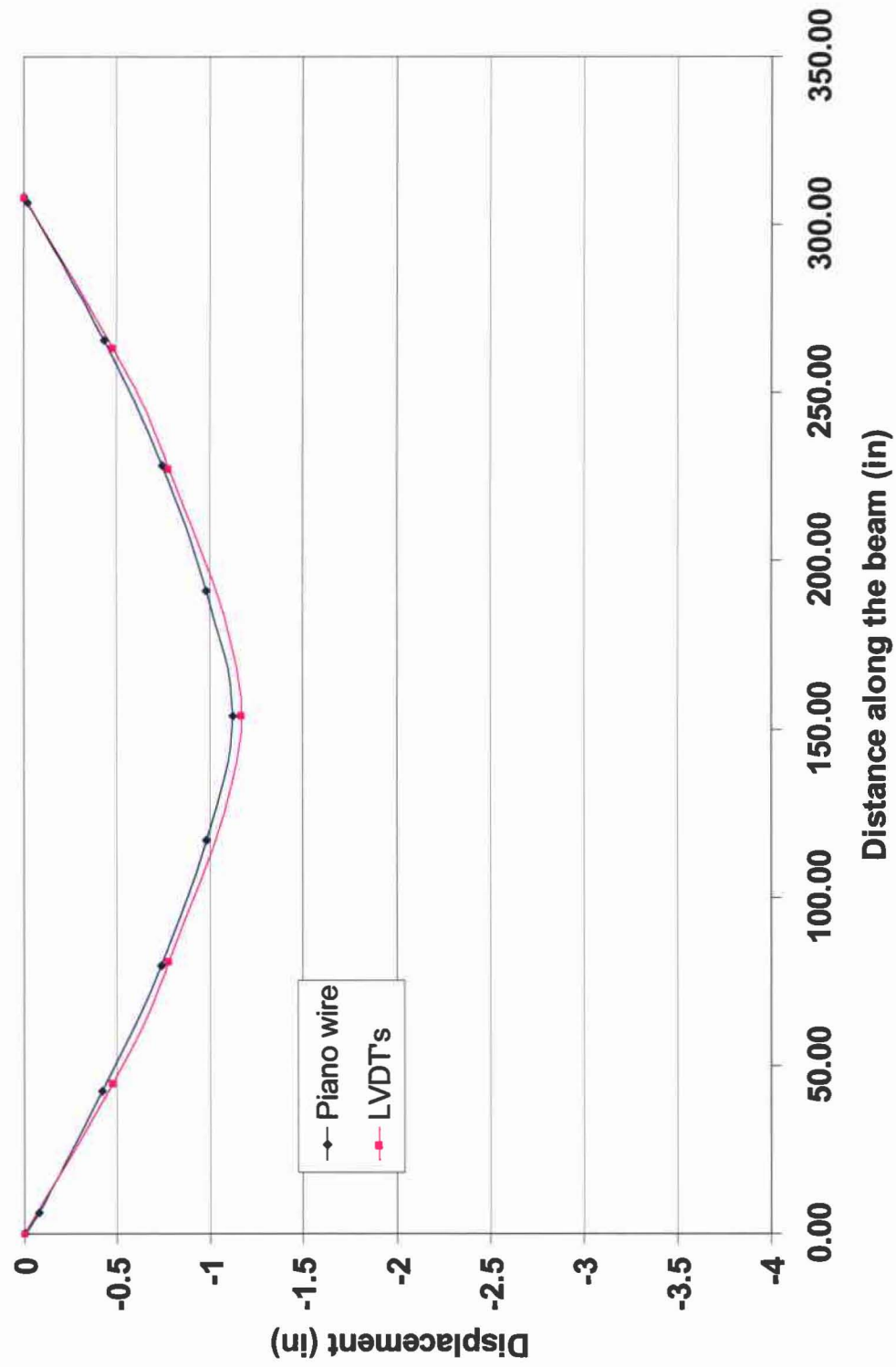
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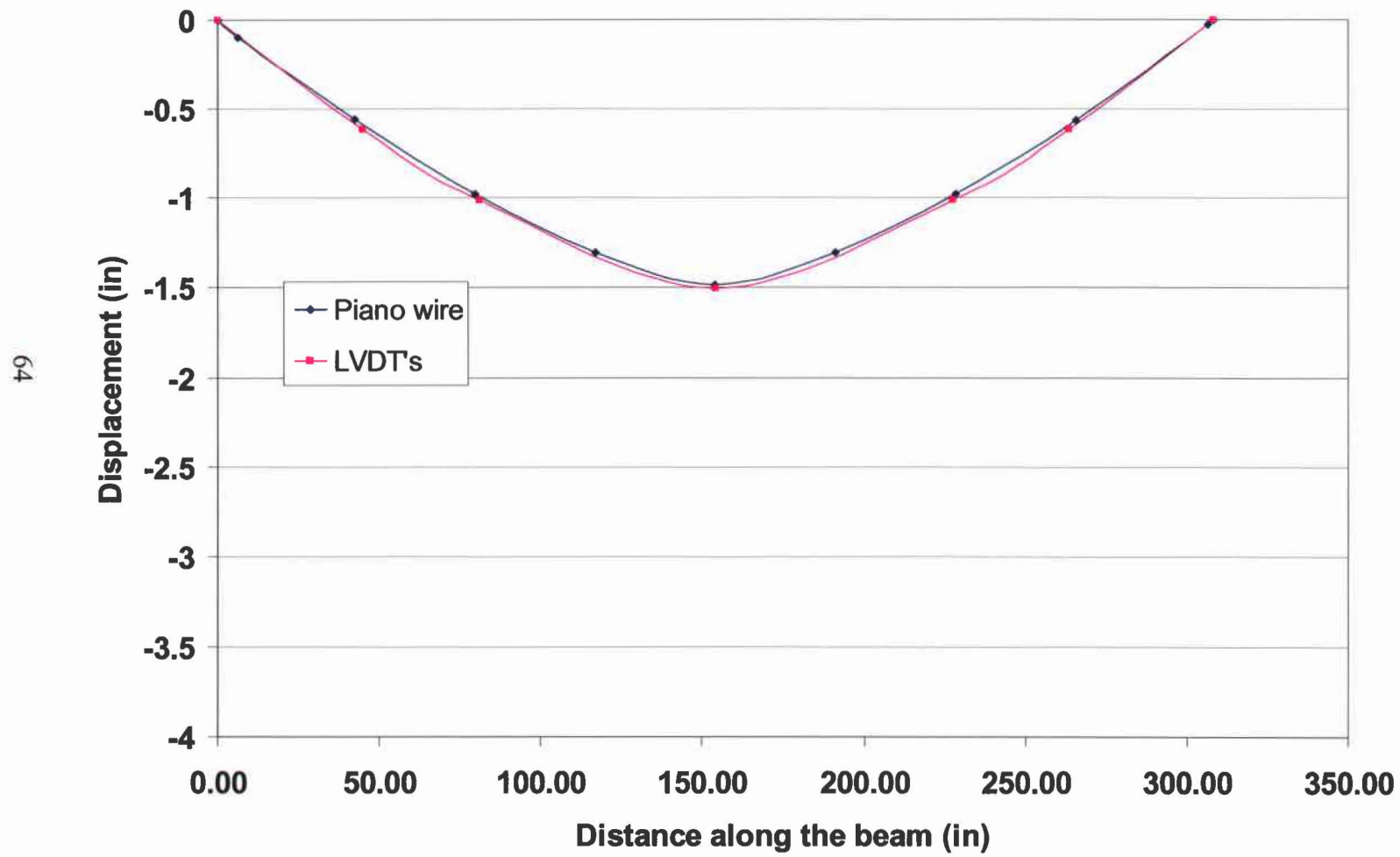
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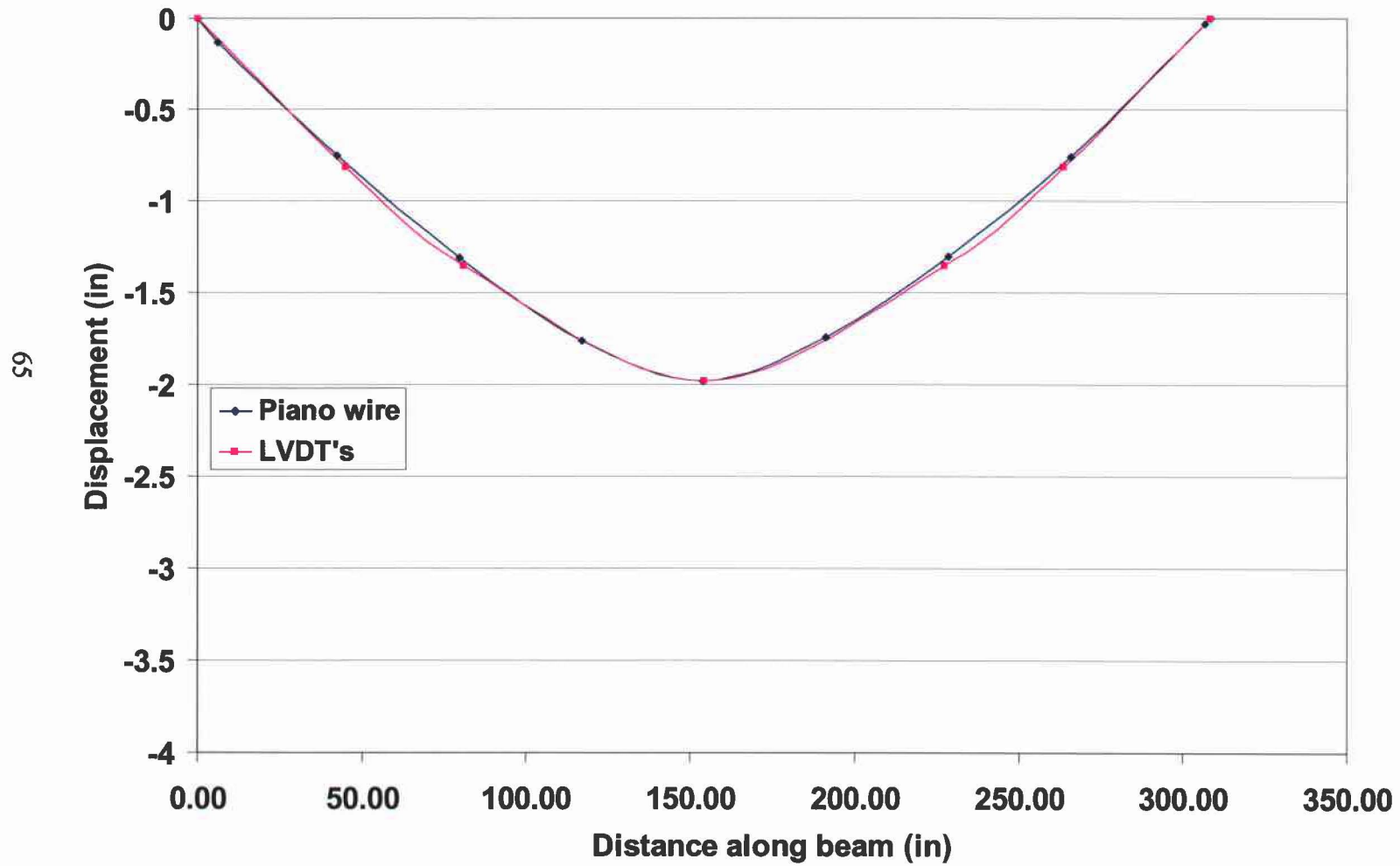
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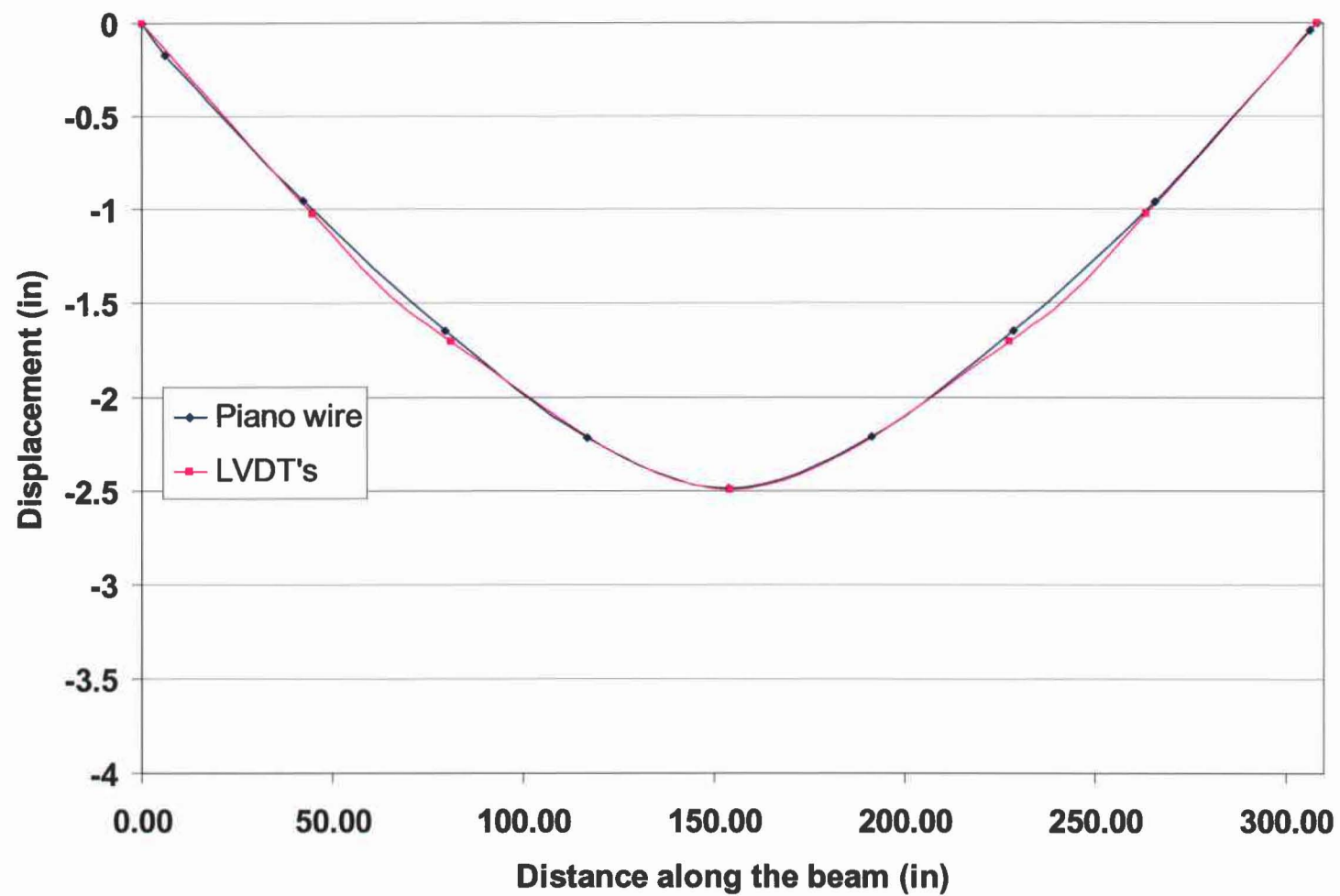
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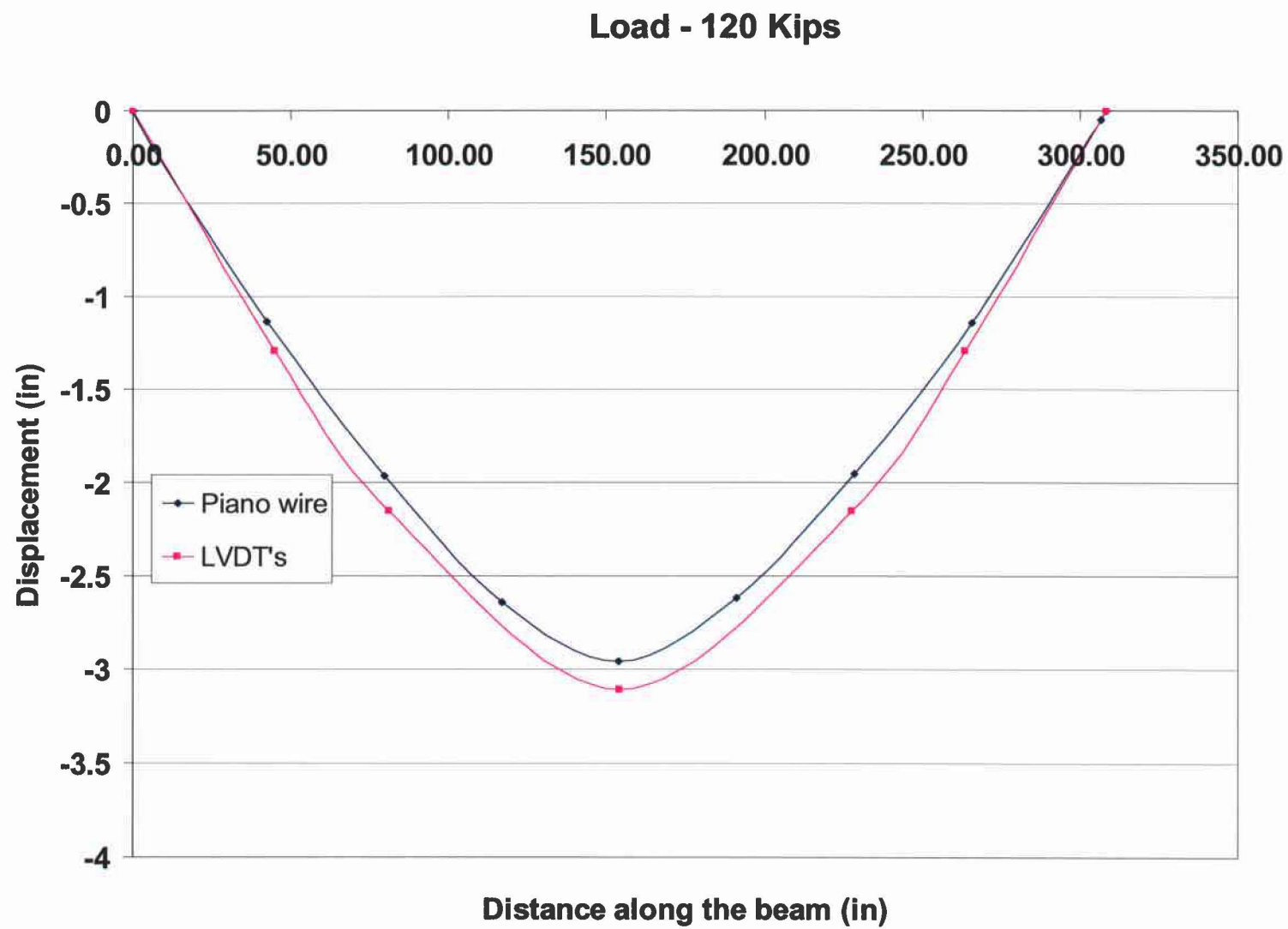


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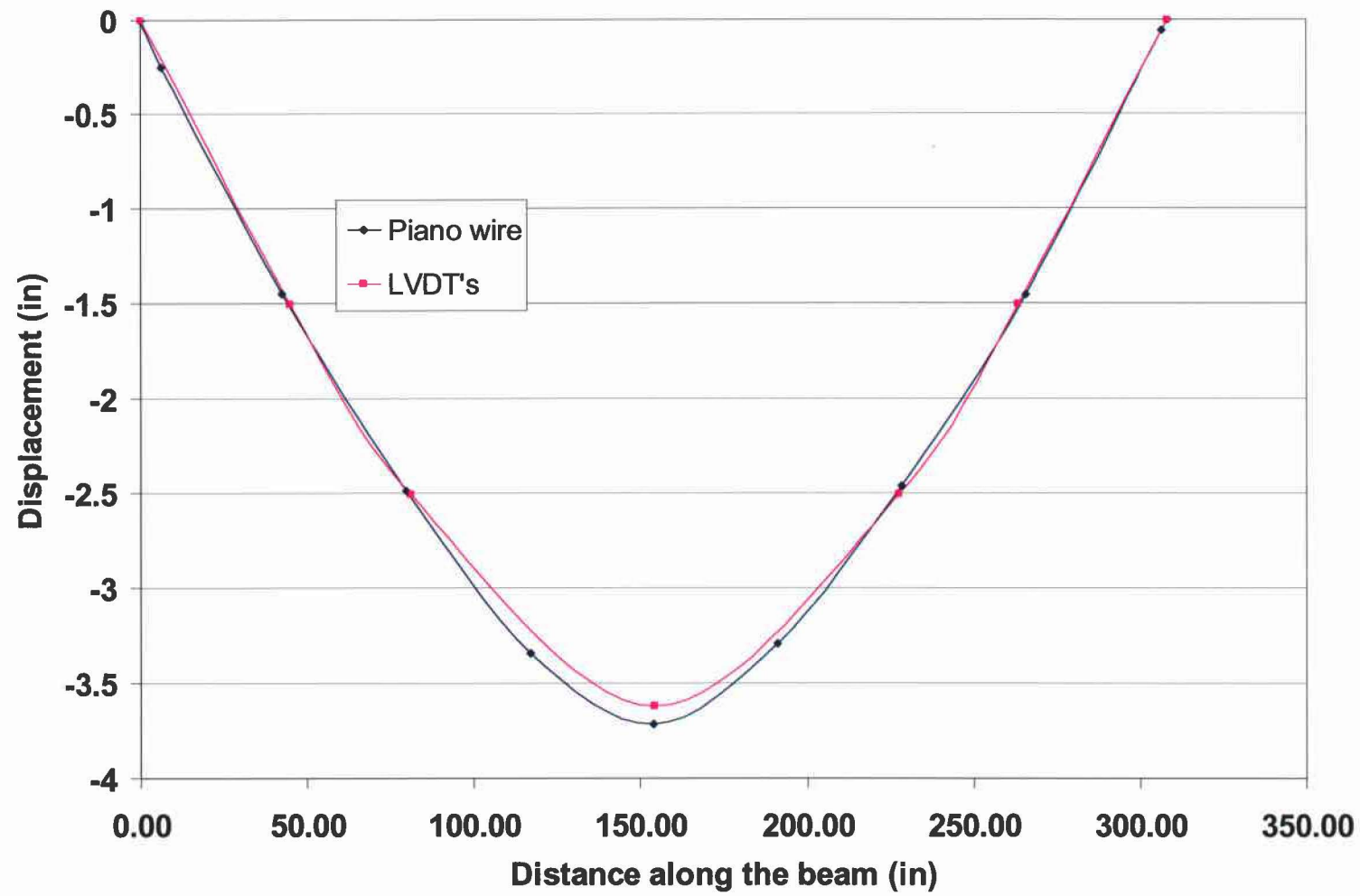


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