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DAMAGE INVESTIGATION USING LONG-TERM MONITORING AND RETROFIT CONCEPT DEVELOPMENT FOR A HIGHWAY TRUSS BRIDGE STRUCTURE IN THE USA

MONITORING, OCENA PRZYCZYN USZKODZEŃ ORAZ KONCEPCJA NAPRAWY KONSTRUKCJI MOSTU KRATOWNICOWEGO AUTOSTRADOWEGO W USA

Streszczenie Rozważany obiekt mostowy leży w ciągu jednej z najważniejszych autostrad w USA. Jest to most kratownicowy stalowy z żelbetową płytą pomostu, jazda górą. Most składa się z 13 przęseł o rozpiętościach od 50 do 200 metrów, w tym 11 przęseł nurtowych; długość całkowita wynosi około 1500 metrów i wysokość około 80 metrów. Most ma około 45 lat i w trakcie użytkowania nie przeprowadzano gruntownych remontów tego obiektu. Inspekcja podpór nurtowych w 2005 roku wykazała znaczne ich uszkodzenia poniżej poziomu wody. Układ uszkodzeń betonowych podpór, głównie w postaci rys i szczelin o rozwartości do 10–15 mm, wskazuje na istotny wpływ konstrukcji pomostu na przeciążenia podpór siłami poziomymi, podłużnymi. Zdecydowano zainstalować na moście, na okres 6 miesięcy, tensometry pomiarowe oraz mierniki relatywnych obrotów i ogniwa termiczne. Wyniki testów wskazują na nie odpowiednią pracę niektórych elementów konstrukcji mostu w korelacji ze zmianami temperatury zewnętrznej i obciążeniem dynamicznym ruchem.

Abstract The case bridge is located on one of the most important of US interstate highways. The bridge is a steel deck truss structure with 13 spans, carrying six lanes of high volume traffic over a waterway channel. The bridge was built in the sixties of 20th century and has never had a major retrofit. Underwater inspection of the channel piers in 2005 concluded that there is significant cracking of reinforced concrete piers up to 10–15 mm in width. The crack pattern suggests that horizontal longitudinal forces from superstructure might have caused these failures. It was decided to install a long-term monitoring system, equipped with strain, rotation and temperature sensors to study the response of the bridge to thermal and live load effects. Monitoring results suggest that temperature gradients and dynamic traffic live load effects may have caused the substructure observed damages.

1. Introduction and Structure Description

The United States interstate highway system underwent major development in the sixties of the 20th century. In particular, a large number of the bridges were constructed in the sixties and are now approaching the end of their 50-year service design life without ever having undergone a major structural retrofit. In the United States, a structural retrofit is commonly used as a short-term solution that will ensure the necessary safety of the traveling public at the lowest cost possible until the structure is replaced. Major structural retrofits are often avoided because of high degree of difficulty in construction with an in-service structure and costs associated with the retrofit, could in many cases equal the complete replacement cost. Therefore, it has been common practice by United States bridge owners and bridge engineers to deploy short-term retrofit solutions and wait for the federal funding necessary to replace the bridge. However, with the current economic situation, they are being forced to change their mind-set and re-evaluate existing structures with the realization that their bridges may not be replaced for another 50-year service life. The owner for the case bridge presented in this paper has realized that the bridge may not be replaced and has deployed measures to prolong its service life with the assistance of consulting bridge engineers. Biannual inspection reports have kept the owner informed of the condition of the bridge and have reported its most severe problem locations. The owner hired the consulting bridge engineers to evaluate the cause of the problem locations and develop a long-term (major) retrofit scheme to keep there bridge safe and operable for another full service life.

The bridge is a steel deck truss structure carrying six lanes of high volume interstate highway traffic over a large waterway channel on the east coast. The bridge has an overall length of 1500 m and is comprised of ten continuous truss spans (including suspended and cantilevered portions between expansion joints), varying from 110 to 150 m in length, and three steel beam approach spans, each 6 m in length. The continuous truss spans are up to 20 m deep and rest on 30 m tall concrete column piers that are supported by large spread footings (10 to 15 m deep) resting on a subfooting foundation founded on bedrock. The superstructure consist of steel floorbeam-stringer system supporitng composite reinforced concrete deck. The column piers consist of two slender rectangular reinforced concrete columns 3 by 3 m average, braced at the level of truss bearings with reinforced concrete strut and with concrete footing at the bottom. The rectangular footings are average 20 m high and have dimensions of 8 by 20 m in plan view. The elevation of the roadway is 80 m from the bottom of the spread footings (60 m above water) [1]

The case bridge was built in early sixties of 20th century and has remained in-service without ever being completely closed to traffic and without any major retrofits. Biannual inspections currently state the bridge as being in overall good condition. The superstructure of the bridge including the steel truss, steel floor system and the reinforced concrete deck all show localized sings of deterioration that are common with an aging bridge, but overall, these elements are in good condition. The substructure of the bridge including the reinforced concrete piers, spread footings and the subfooting foundation are in worse condition, particularly the spread footing and subfooting foundations. Underwater inspections of the substructure in the eighties of the 20th century found numerous large horizontal cracks in the spread footing and through the entire width of the subfooting foundation all piers [2]. As a result, measures to repair and cease further cracking were taken by encasement of several spread footing and subfooting pier foundations with tremie concrete. Observations of further cracking and reopening of existing cracks after the substructure repairs led to a 2nd attempt to allieviate the cracking in the nineties of the 20th century. In this 2nd repair attempt, existing substructure cracks were epoxy injected and the superstructure floorbeam bearings were replaced. 2006 Underwater inspections have identified further degradation of the tremie concrete and further opening of the full width horizontal cracks throughout pier foundations. Conclusions from the underwater inspections suggested possible pier movement based on the uneven vertical faces of the foundation concrete at the location of the horizontal cracks and recommended immediate substructure damage investigation and retrofit. Consulting bridge engineers were hired to develop long-term substructure retrofit concepts and to evaluate the current level of substructure damage. After preliminary analysis and evaluation of possible causes for the substructure damage, it was decided to perform long-term monitoring of the bridge to better understand the in-service behavior of the structure and to verify preliminary assumptions as to the cause(s) of the substructure damage. The long-term monitoring was aimed at studying the response of the bridge to traffic live loads and temperature gradients.

The remaining sections of this paper provide further detail on the prior bridge inspections and their findings, the preliminary analysis of the bridge condition, the long-term bridge monitoring system, results from the long-term bridge monitoring and conclusions.

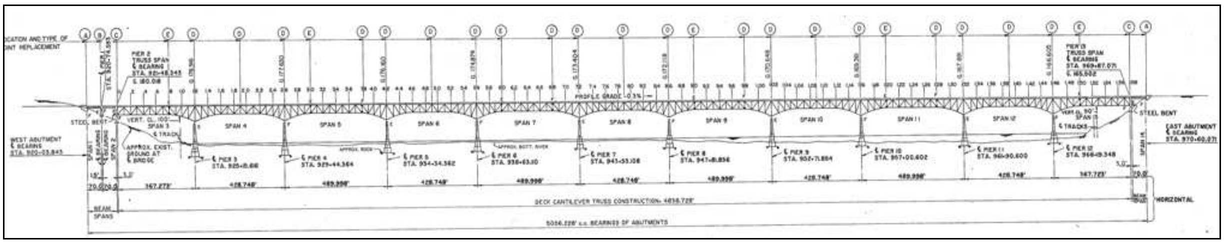

Figure 1. Bridge Plans – Elevation view

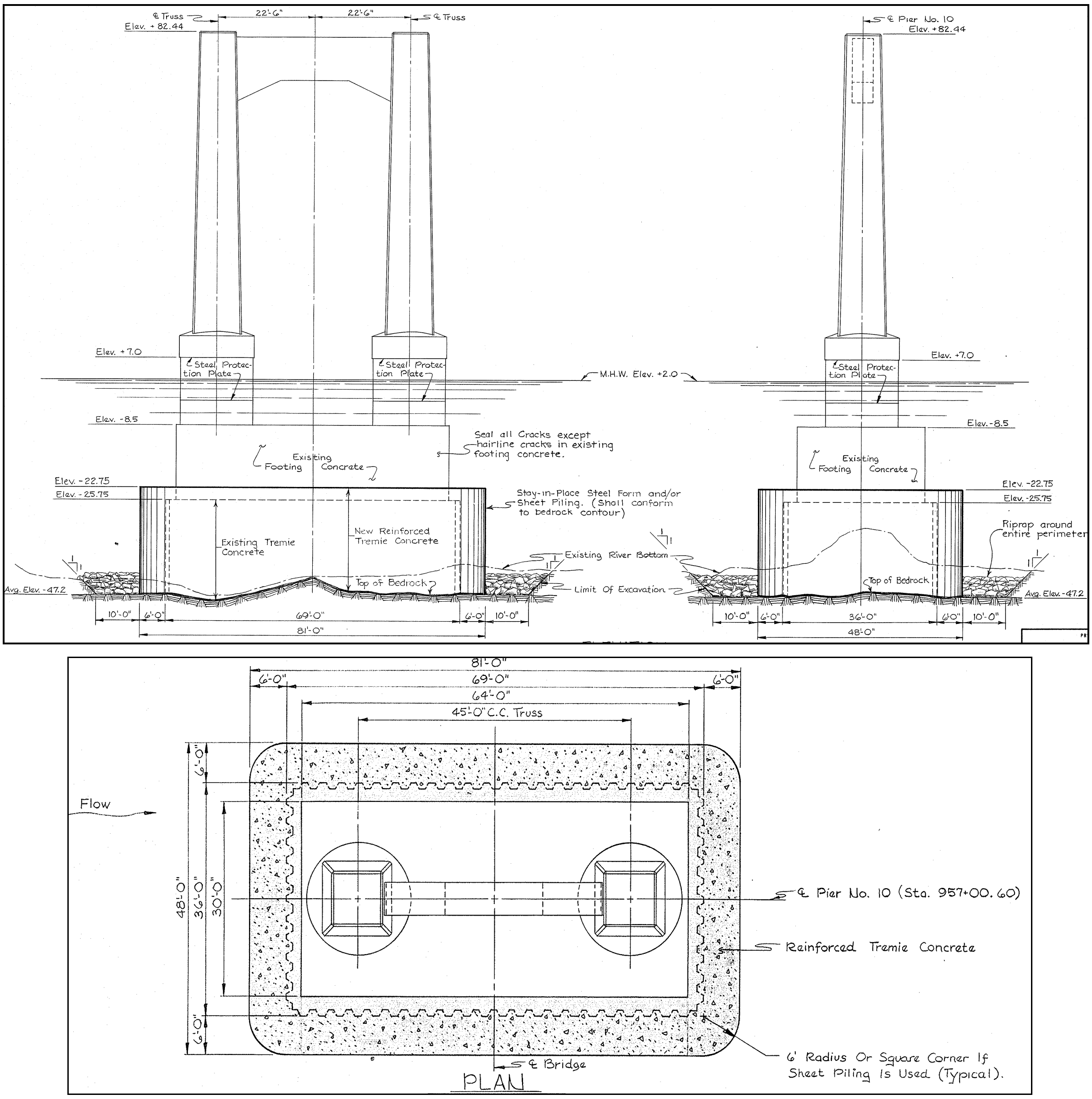

Figure 2. Bridge Plans – Longitudinal and Transverse Elevation and Plan View of Typical Channel Pier

Figure 3. Elevation view of inspected bridge

2. Prior Inspections and Their Findings [2]

Biannual inspections reported the bridge substructure (piers 2 through 11) as being the most problematic locations on the bridge. Each pier consists of two circular reinforced concrete columns supported by spread footings with a subfooting founded on bedrock. The concrete columns are up to 5.2 m in diameter and have a sacrificial steel encasement in extending for 3.0 m in the tidal zone of each column. The two columns are supported by a reinforced concrete spread footing that rests on a reinforced tremie concrete subfooting formed with steel sheetpiling. Piers 8 through 12 contain the additional reinforced concrete tremie encasement from the repairs in the eighties of the 20th century.

The most recent (2005/2006) underwater inspection of the bridge piers (piers 2 through 12) revealed submerged substructure conditions, as described below.

Pier columns were found to be in fair condition with concentrations of hairline vertical cracks found in all columns on the portions nearest to the footings edge. These cracks were typically around 1.5 mm wide with one crack at pier 11 opened to a width of 4.5 mm. The sacrificial steel encasement exhibits light corrosion and roughly 30% section loss. The crack pattern of pier columns is coincident with the pattern on spread footings and subfootings.

The spread footing at pier 3 is not visible due to high mudline at this location. The spread

footings at Piers 4 and 5 are partially exposed and are in fair condition. Spread footings at Piers 6 through 12 are in poor to serious condition due to extensive horizontal and vertical cracking. Overall, less than 50% of the observed cracks have undergone repair and a majority of the repaired locations are experiencing further cracking due to failed or failing repairs. In particular, the most concerning cracks observed were previously repaired horizontal cracks that were epoxy injected in the nineties of the 20th century and have since reopened significantly. In pier 6 for example, an epoxy injection repair was done for a 13 mm wide horizontal crack in the nineties and the most recent inspection found the epoxy material in-tact, but now the horizontal crack opened an additional 6 mm. The total crack opening is thus around 20 mm. Similar conditions occurred in numerous locations, with horizontal shift of concrete below the crack in reference to concrete above.

The original tremie concrete subfootings are only exposed at piers 6 and 7 and the encased tremie concrete subfootings are partially exposed at piers 8 thru 12. Pier 7 is the most exposed subfooting with only 1.0 m of visibility from the top of the subfooting. The subfooting of pier 7 is assumed to be representative for all the piers and is considered to be in poor condition. The concrete is primarily latent and appears to have been dropped in place rather than constructed with actual tremie pours. Intermittent latent pockets are up to 10 cm deep. Only one pour joint is visible and that joint has opened between roughly 13 to 20 mm. Latent concrete is present below the pour joint and honeycombing is present above the pour joint. Additionally, the subfooting section below the pour joint has shifted outward in several locations. There are several large vertical cracks on one face of subfooting that are opened up to 16 mm wide and continue below the mud line, with evidence of movement and rotation.

In comparison to the earlier 2000 underwater inspections, the quantity and size of cracks in the 2006 underwater inspections have increased significantly in all portions of the substructure. The 2006 underwater inspections strongly recommend a thorough engineering analysis of the pier cracking and a long-term retrofit of the substructure for this bridge.

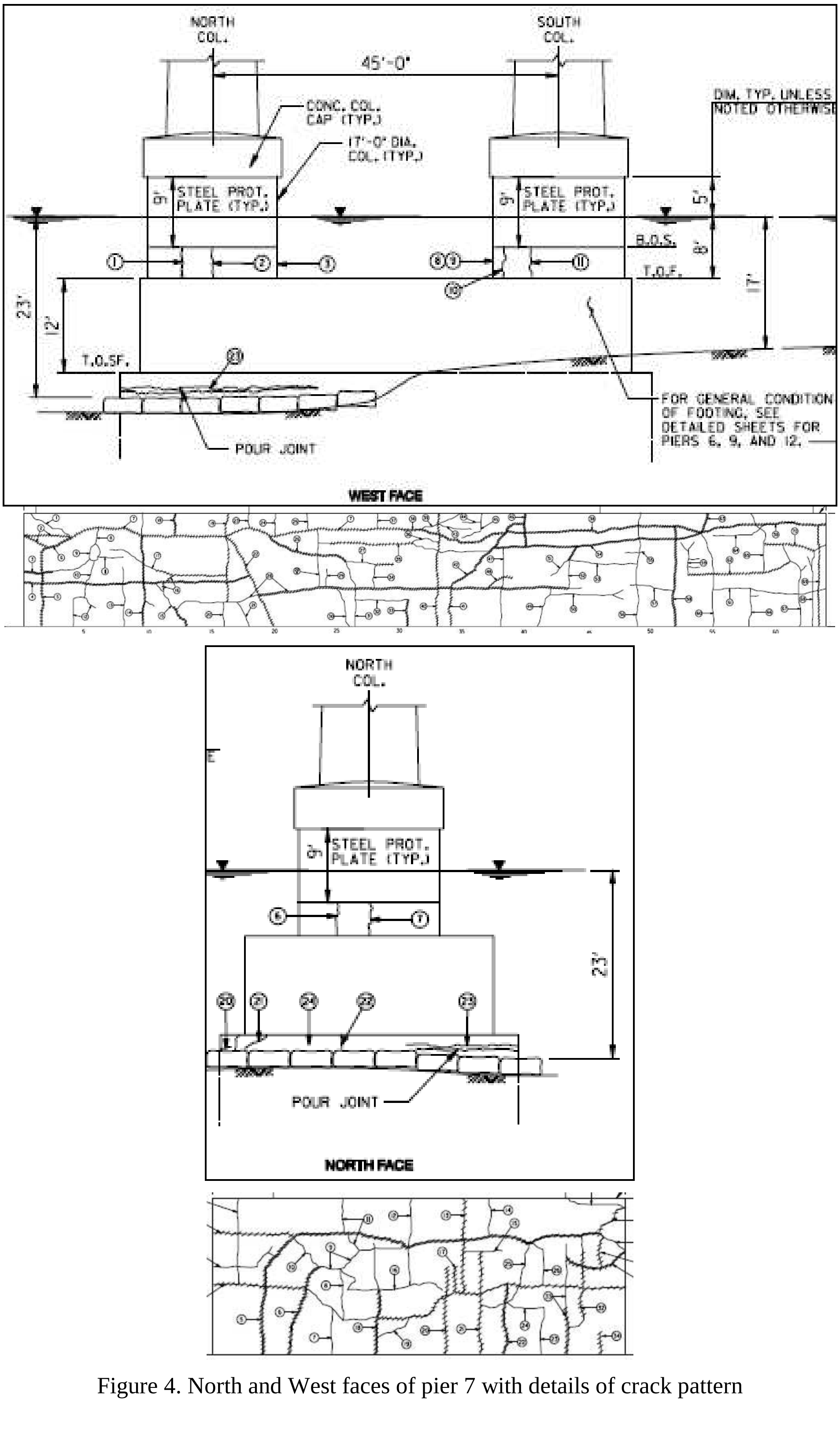

Figure 4. North and West faces of pier 7 with details of crack pattern

3. Preliminary Analysis of Bridge Condition

The history of the biannual inspection reports were used to predict causes of the pier foundation failures. A summary of the hypothesized causes of failures are presented below.

The observed map cracking of the subfooting tremie concrete, spread footing concrete and lower portions of the concrete columns is likely due to the effects of heat released during the early hydratation process from when the foundations were first constructed. Differential thermally induced stresses in an under-reinforced to lightly reinforced mass of concrete, such as all three of the foundation components, would cause the random vertical and horizontal hairline cracks similar to the map cracking observed. Core samples taken from the subfootings indicate large pour depths that would make the subfootings susceptible to temperature and shrinkage cracking. Additionally, the concrete mixtures used in the subfootings were observed to be non-uniform in gradation and compressive strength with large amounts of latent tremie concrete along the pour joints. The map cracking on the spread footings and lower portions of columns contained larger openings at the foundation face that became progressively smaller in width and nearly hairline at the level of reinforcement. More extensive map cracking was observed in regions of little to no reinforcement. The observed cracking of the spread footings and lower columns is likely due to old reinforcement details that were not well suited to mitigate the thermal stresses from the hydration process.

The large horizontal cracks throughout the spread footing and subfooting portions of the foundation are believed to have developed from an unintended response of the structure due to either live load or thermal effects. These large cracks were repaired with a reinforced tremie concrete encasement in the 1980's and also epoxy injected in the 1990's and were found to have re-opened in the most recent underwater inspections. The 2006 inspection also suspects lateral movement of the pier foundation because of a visible misalignment in the vertical plane of the foundation faces for cracked sections. If lateral movement of the pier foundation has occurred it could have resulted from an unintended large lateral force produced from the superstructure and transmitted into the foundations through the tall piers.

The consulting engineers decided to perform, in the first phase of retrofitting concept development, a superstructure inspection as well as stress and deformation state monitoring of piers. The inspection included a detailed record of the behavior of all expansion joints and rocker supports for different temperatures. Stress and deformation state monitoring was performed on a representative pier using a long-term data acquisition system to collect data on pier temperature, pier strains and pier rotations as well as bottom chord truss temperature and strains. Details of the long-term monitoring system are provided in next section.

4. Long-Term Bridge Monitoring System

The long-term monitoring system was installed on the case bridge in July of 2008. Pier 9 was selected as the representative pier foundation for the bridge. The superstructure inspection of expansion joints and rocker bearings provided confidence that any unexpected behavior noticed in pier 9 would be properly transferred to other piers of the bridge.

The long-term monitoring system consists of a data collection unit (data logger), 18 strain sensors (strain gages), 6 rotation sensors (tiltmeters), 4 temperature sensors (thermocouples) and a battery bank power source with daily charging from two 75 Watt solar panels, see Figure 5. Data is transmitted to a static IP address through a wireless modem with antenna and is collected daily.

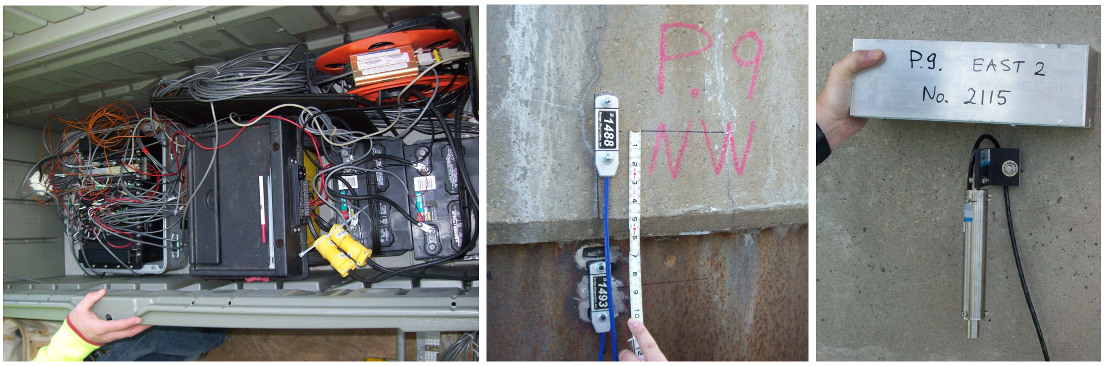

Figure 5. Monitoring system: data logger, strain gage and tiltmeter

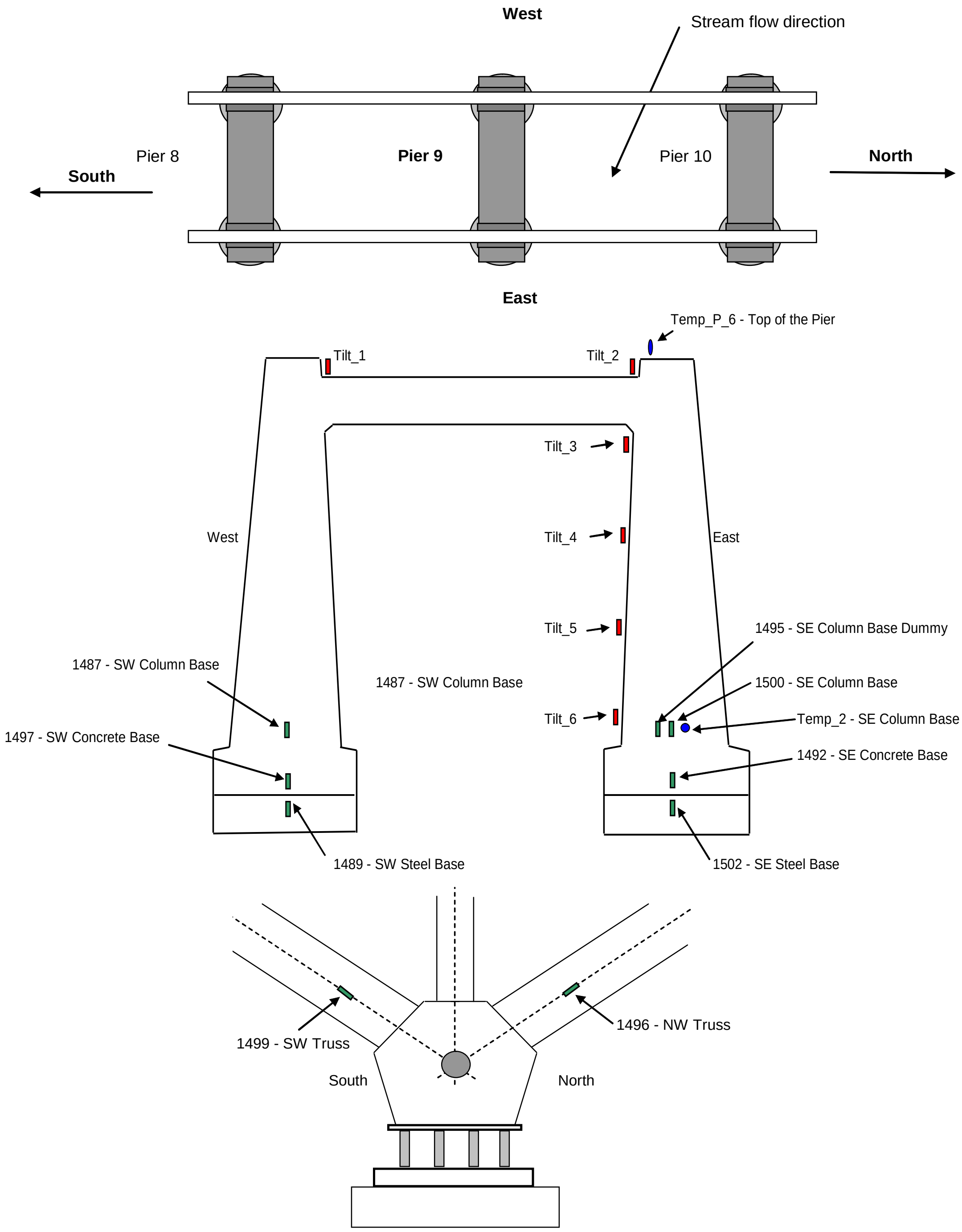

Figure. 6. Bridge acquisition system setup: top view of piers 8 thru 10, side view of pier 9 and rocker bearing

The data logger was installed at the top of the pier 9 and connected to sensors using the wire. Strain gages were installed on: 1. the centroid of the web for each bottom chord truss member framing into the rocker supports of both trusses; 2. each pier column at three different elevations on the lower portion of the columns, and on opposite sides of both columns. Also, two dummy gages were installed on one of the columns on opposite sides and close to actual gages to quantify the effects of ambient noise in measurements. Tiltmeters were installed to the face of one column, throughout its height to measure pier rotations in the longitudinal axis of the bridge. Thermocouples were installed on the structure close to gage locations to measure material temperatures with one sensor for measuring air temperature. A diagram with sensor locations on pier 9 is shown in Figure 6.

In order to minimize postprocessing efforts and to maintain representative measurements, it was decided to take sensor measurements for one minute at the top of each hour at a frequency of 20 Hz and collect the extremes (maximum and minimum) as well as the average measurement from the 1200 measurements taken per sensor. This collection strategy captured the effects of both temperature gradients and traffic live load in the measurements.

5. Results of Long-term Bridge Monitoring [3]

Monitoring of the bridge has been ongoing since July of 2008. Important findings about the response of the bridge in terms of deformations and stresses have been observed from the collected data. For the purpose of this paper, only two weeks of selected data will be used to explain these important findings.

The collected data includes temperature, strains and rotations of the instrumented locations on pier 9 and the lower chord truss members framing into the supports of pier 9. Data analysis first focused on isolating the temperature effect from the measured strains and rotations. Figure 7a illustrates daily temperature gradients with a maximum gradient of roughly 23° F for air temperature and 30° F for structure temperature between the 6th and the 10th day of data sample. Minimum temperature gradients, below 10° F, were between 2nd and 5th day of the data sample. The maximum temperature gradient in the sample would cause an elongation of steel members up to 0.02% and up to 0.017% for concrete members. An arbitrary 10 meter long steel member would experience a 2.0 mm elongation if unrestrained and, if restrained on both ends, 60 MPa of normal stress. A corresponding concrete member would experience 1.7 mm elongation and 7 MPa normal stress. This is a significant amount of normal stress considering that concrete tensile strength of is roughly 4 to 6 MPa and the yield strength of steel is near 450 MPa.

Measurements from a selected tiltmeter are presented in Figure 7b. The selected tiltmeter is located at the top of the pier and measurements are in terms of relative rotation in the longitudinal direction of the bridge. Average rotational gradients were found to be relatively small and when compared to the plots of the temperature gradient, a clear trend exists suggestive that the rotational position of the pier conforms to the effects of superstructure elongation from thermal gradients. The extreme (maximum and minimum) measurements were observed to have greater short term effects on pier rotation than thermal effects. The extreme measurements are believed to have been caused by the short term live load effect. The maximum average rotational gradient (thermal) was observed on day 7 to be around 0.06 degrees, corresponding to 1.2 cm of longitudinal translation at the top of the pier. The maximum extreme gradient (live load) was observed to be around 0.9 degrees, corresponding to 15 cm of longitudinal translation at the top of the pier. The rotational response of the pier is reasonable. Pier 9 experiences some relatively insignificant bending due to a slow thermal effect.

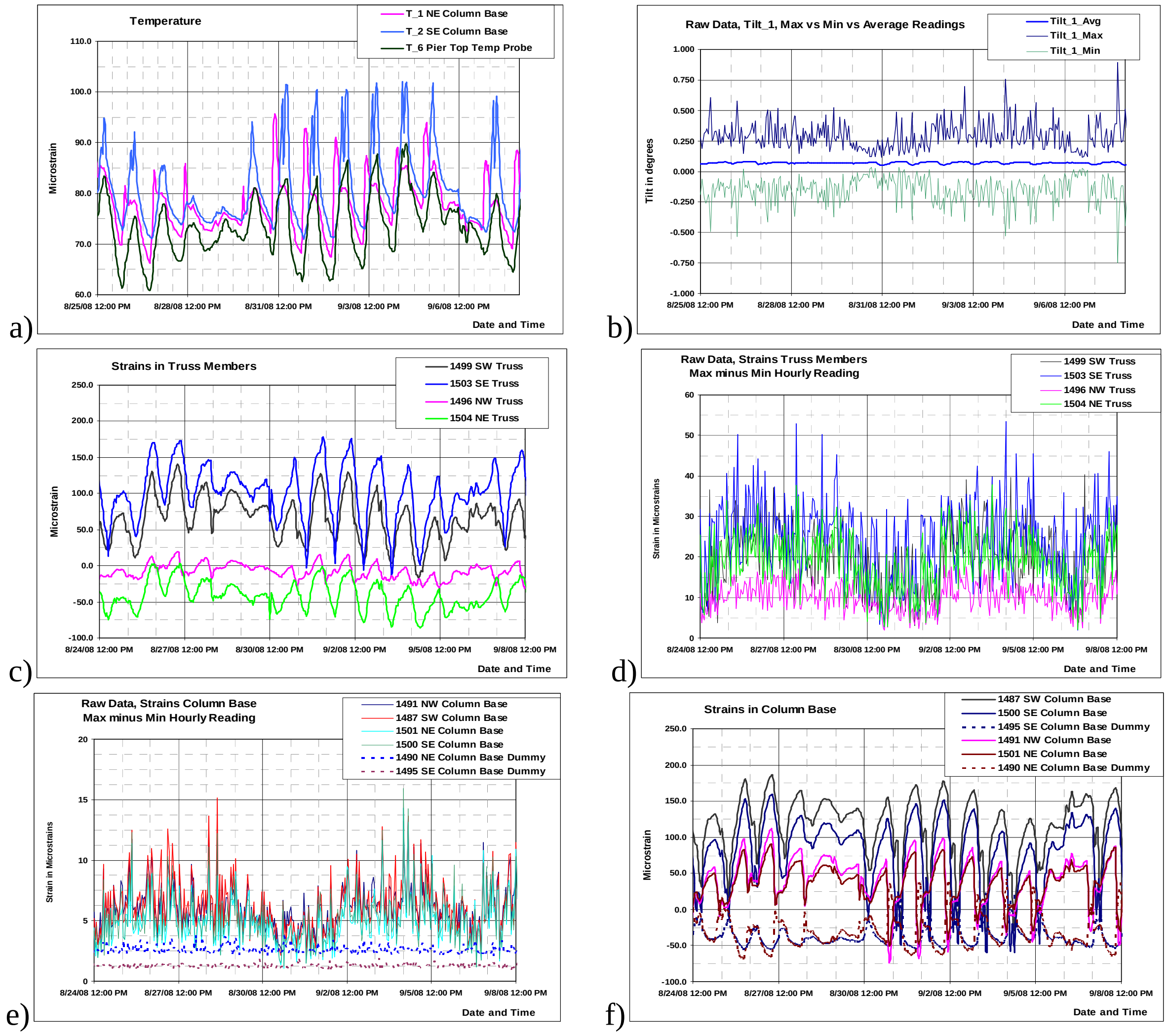

Figure 7. Readings from thermocouples, tiltmeters and strain gages on truss members and steel columns

The expansion joints and the truss rocker bearings work as designed and allow for nearly free movement. Pier 9 experiences significant bending, most likely due to the frozen response of the rocker bearings, when subjected to high frequency live load changes and dynamic effects. Tiltmeter data for extreme measurements (live load) also illustrated weekly patterns. Larger values of measured rotations were recorded during week days rather than weekends, see days 5–7 and 12–14. This trend is common of truck traffic and serves as good support for crediting extreme measurements to be caused from live load effects.

The trends observed from the tiltmeter response are not confirmed by response of strain gages. The strain gage measurements allow for estimating strains, ans based on these stresses and global forces which, in theory, could be related to the measured rotations of pier 9. Figures 7c and 7d illustrate strains in the bottom chord truss members. Figure 7c illustrates average strain readings while Figure 7d illustrates the difference between maximum and minimum readings (the strain gradient from one minutes worth of samples (1200 measurements)). Daily strain gradients from thermal effects (maximum of 180 με, corresponding to 53 MPa of normal axial stress were observed to produce much larger strain gradients than those due to short-term live load (maximum of 54 με corresponding to 15 MPa of normal axial stress. When comparing measured strain gradients in bottom chord truss members from both each side of the rocker support, it was observed that the maximum difference between daily strain gradient from thermal effects is roughly 100 $\mu\varepsilon$ (2,9 MPa) and from live load effects is roughly 22 $\mu\varepsilon$ (0,65 MPa).

6. Conclusion

The long-term monitoring results show that horizontal forces at the top of the piers do exist and that their effects are experienced at the pier foundations. It is possible that these horizontal forces in combination with the poor condition of the substructure foundation concrete may have caused the extensive stress cracking observed. Pier rotation measurements and field inspections data suggest that the thermal effects on the bridge are dissipated quite well though the expansion joints and rocker bearings while live load effects are absorbed by the pier due to frozen bearing behavior when subjected to short-term dynamic loads.

The true nature of the governing horizontal forces must be studied through further monitoring and evaluation. Upon further identification of reasons for the substructure failures a long-term retrofitted solution will need to be developed. Monitoring efforts will continue until March of 2009. For now, it was determined that both temperature gradients and live load contribute to accelerated degradation and damages of pier foundations.

Literature

1. Bridge Original Plans *
2. Inspection Reports from 2000, 2005, and 2006 *
3. Prelimanary Monitoring Report *

* Bridge Owner did not allow to release any more specfic data and information to public.