PILE DRIVING EFFECTS ON A STEEL TRUSS RAILWAY BRIDGE DURING REHABILITATION

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Abstract

This paper summarizes the performance of a steel truss railway bridge near Saskatoon, SK, which remained in service during pile driving activities for pier rehabilitation and new pier construction.

289 H section piles were driven to embedment depths of 9 m for rehabilitation of existing concrete piers, and 12 m for new piers, for the western portion of the bridge, over an approximate one-month timeline. Piles for existing piers were driven within one metre of the existing pile caps, which were supported on timber piles. Monitoring instrumentation included surveying of prisms mounted to the bridge deck and piers and installation of tilt loggers, and a vibration monitoring system, to monitor the lateral deflection and accelerations, respectively, of the structure during pile driving. The collected data provides an understanding of the response of the bridge structure from pile driving into the hard foundation till, along with expansion and contraction effects due to extreme temperature variations. Survey and tilt logger data were found to correlate well together, and with changes in ambient temperature.

Wave Equation Analysis of Pile Driving (WEAP) was conducted to estimate pile termination criteria and driving hammer performance. Pile Dynamic Analyzer (PDA) testing was conducted on 10% of the piles; 9 and 12 m long piles driven into Sutherland Till exhibited average vertical capacities in the range of 1,940 kN and 2,700 kN, respectively.

1.0 Introduction

1.1 Background

The Canadian National Railway (CN) crosses the South Saskatchewan River at Mile 60.8 on the Aberdeen Subdivision approximately 7.5 kilometers east of Warman, Saskatchewan, (Figure 1) on a streel truss bridge founded on concrete piers supported by timber piles. Construction and design details for the timber piles are not known. The bridge was scheduled for rehabilitation, with construction starting in the fall of 2017, and completion in early 2019. Rehabilitation included the installation of over 500 driven H section steel piles as foundation support to retrofit existing piers for increased capacity, and for construction of new piers. Information in this paper is based on the work completed on the western portion of the bridge, and examines the predicted performance of the new piles based on WEAP, compared to the as-constructed capacity of the piles from PDA testing. Observed effects on the bridge as a result of the pile driving are discussed.

1.2 Geologic Setting

The geology of the Saskatoon area is described by Christiansen (1968) and Christiansen (1970). A series of glaciations during the Pleistocene epoch deposited, in descending order: surficial stratified drift; Battleford Formation till; Floral Formation till; Sutherland Group till. Pre-glacial sediments underlie the Sutherland Group till: discontinuous Bearpaw Formation silt and clay, Oldman Formation sands and silts; Lea Park Formation – Upper Colorado Group silt and clay.

The South Saskatchewan River has eroded into the Sutherland Group till, which was the load bearing strata for the existing and rehabilitated bridge foundations. The stratigraphy of the river bottom consisted of fluvial granular depositions (river sands and gravels), overlying an eroded boulder lag, with boulder diameter ranging from approximately 200 mm to 1000 mm, with an average size in the range of approximately 400 mm. Sutherland Group Till was present beneath the boulder lag, which was comprised of a clay matrix with varying proportions of silt and clay; in general, silty sandy clay with a trace of gravel and cobbles, and occasional boulders. The undrained shear strength of the Sutherland till was measured as high as 720 kPa, with an average of approximately 500 kPa, based on laboratory compression testing.

2.0 Construction Setting

To provide access to the western portion of the bridge, an access road was excavated into the west valley wall, and a cofferdam constructed in the river around the existing three westernmost piers. The existing piers, from east to west, were designated Piers 6, 8, 10 and 12, with Piers 6 through 10 located within the river, and Pier 12 located on the valley wall. New piers were designated Piers 7, 9, and 11.

The floor of the cofferdam was constructed below the elevation of the river bottom, at an elevation of approximately 461.5 m, and the top of berm at 466.3 m. The boulder lag was removed so that the piles could be driven from the Sutherland till surface.

Existing piers were approximately 17 m in height from the top of pile cap to top of pier, and approximately 20 m long at the base, 9 m long at the top, and 4.5 m wide at the base. Pier pile caps were approximately 20.5 m long and 7.5 m wide. The steel truss section was approximately 10 m high and 5 m wide. Figure 3 shows piling underway at Pier 7.

2.1 Foundation Design

289 piles were installed at Piers 6 through 11 over the course of approximately one month in November and December of 2017. The design loading for piles for the existing and new piers was 570 kN and 1,080 kN, respectively, corresponding to a required factored geotechnical resistance of 1,140 and 2,160 kN, respectively, using a reduction factor of 0.5.

Steel H section HP310x110 piles were used for new and existing piers. Rehabilitation of existing piers included the installation of 17 - 9 m long piles along each side, which were driven to approximately 8.3 m. Pile caps were cast to the new piles and mated to the existing pier pile caps with Dywidag bars.

Foundation design for the new piers included 57 - 12 m long piles, driven from excavations approximately 2 m below the cofferdam floor to provide clearance to construct the pile caps in similar fashion to the existing piers. Piles for new piers were driven to approximately 11 m.

2.2 Wave Equation Analysis of Pile Driving

WEAP was undertaken to estimate pile driveability, the suitability of proposed driving hammers, and termination criteria. Foundation design parameters are shown in Figure 2. Hammers considered included: Junttan HHK3A, HHK4A, HHK5A and HHK5S.

The HHK5A hydraulic hammer was recommended and used. Hammers with lower energy ratings were not expected to provide enough impact force to re-mobilize the piles during re-strike. Specifications for the HHK5A were: Energy 58.884 KJ, Ram Weight 49.07 kN, Maximum stroke 1.2 m, Efficiency 0.8. Quake and damping values used were as follows: Quake (shaft and toe): 2.50; Shaft damping: 0.650; Toe damping: 0.50. Quake and damping was distributed uniformly along the shaft.
Tables 1 and 2 show preliminary termination criteria for existing and new piers, respectively. A test pile program was conducted with the first eight piles, after which termination criteria was revised based on as-built conditions. Revised values are shown alongside the preliminary values in Tables 1 and 2. Design capacity was met in some cases during the test pile program after one day, so the PDA testing program was revised, with the minimum requirement of one-day re-strikes to confirm capacity, along with the revised Test Piling Program termination criteria in Tables 1 and 2. Additional re-strikes were conducted if capacity was not met within one day. Eight piles was chosen as an experience based judgement with the intent of collecting enough data to conduct a representative analysis, while proceeding with the relatively tight construction schedule.

3.0 PDA Testing

3.1 Data Collection

Of the 289 piles driven for Piers 6 through 11, PDA testing was conducted on 38, following the project specification of PDA testing on a minimum of 10% of installed piles, and at least one test per structure. Approximately 25 % of the PDA tests were conducted at the end of initial driving (EOID), and 75 % at the beginning of re-strike (BOR). Re-strikes were conducted at one to 10 -day intervals, depending on pile capacities obtained during re-strikes, and construction scheduling. Piles tested at EOID were also tested on re-strike, while piles tested at BOR were not always tested at EOID.

3.2 Analysis

CAPWAP signal matching was conducted for each tested pile to determine capacity using the Case Method, along with distribution of shaft resistance and toe resistance.

Commentary on pile capacities, transferred energy, maximum compressive stresses and pile integrity is provided in sections below.

3.3 Pile Performance: Existing Piers

Figure 4 shows the calculated capacity from the Case Method in CAPWAP for rehabilitated piers. Each point in Figure 4 represents the averaged capacity for all piles driven for existing piers with the same re-strike time. EOID values are shown as zero days. The required factored geotechnical resistance of 1,140 kN was met for most piles at the end of driving, and for all piles after one day. The capacity increase due to soil setup was in the range of 1.3 to 1.5 times the EOID capacity after one day.

3.3.1 Hammer Transferred Energy

The maximum observed hammer transferred energy at EOID and BOR was 50.3 and 63.8 kJ, respectively. These values correspond to a hammer efficiency of 85% and 108 % compared to the rated energy. Reported hammer efficiencies are calculated by the PDA software based on the maximum theoretical energy available for the hammer.

3.3.2 Maximum Driving Stresses

Minimum and maximum driving stresses for existing piers was 158 and 309 MPa, respectively, which was below the recommended driving stress of 315 MPa. Driving stress in Pile No. 416 at Pier 8 was measured at 340.4 MPa during re-strike, which was above the recommended driving stress, but less than the yield stress, which was 350 MPa.

3.3.3 Pile Structural Integrity

All PDA testing showed values for the Beta factor (BTA) greater than 0.8, indicating only slight pile damage. Values of BTA greater than 0.8 are generally acceptable, and were accepted for this project.

3.4 Pile Performance: New Piers

Figure 5 shows the calculated capacity from the Case Method in CAPWAP for new piers. Each point in Figure 5 represents the averaged capacity for all piles driven for existing piers with the same re-strike time. EOID are shown as zero days. The required factored geotechnical resistance of 2,160 kN was often met at the end of driving, achieved in most cases after one day of setup, and achieved for all tested piles after seven days. Setup for new pier piles ranged from approximately 1.15 up to 1.7 times the EOID capacity.

3.4.1 Hammer Transferred Energy

Hammer transferred energy at EOD and BOS was 57.9 and 59.9 kJ, respectively, indicating hammer efficiency of 98% and 102% respectively, compared to the rated hammer energy.

3.4.2 Maximum Driving Stresses

Driving stresses for new piers were measured at a minimum of 225 and maximum of 315 MPa, which was below or equal to the maximum recommended driving stress of 315 MPa.

3.4.3 Pile Structural Integrity

All PDA testing for new piers showed BTA values greater than 0.8.

4.0 Vibration Monitoring

4.1 Instrumentation

The risk of impact to the bridge as a result of pile driving activities was not well defined, so a vibration monitoring program was implemented to monitor accelerations of the piers.

Wilcoxon Research 787A 100mV/g accelerometers were installed in three locations on piers 6, 8 and 10: one installation near the base of the pier, one near the midpoint, and one near the top of concrete. Each installation included one accelerometer each for the x, y and z directions, specified as follows: x parallel to the railway; y perpendicular to the railway; z vertical.

Accelerometers were hard wired to a Pruftechnik Vibguard 20 channel online condition monitoring system, which was installed at the top of Pier 8 to ensure a reliable cellular connection. The monitoring system collected and processed the accelerometer data, which was automatically sent to a remote server via cellular modem. Pruftechnik Omnitrend software was used to complete analysis: an example of typical processed vibration monitoring data is provided in Figure 6.

Limits for vibration monitoring were provided by CN:
- <10 mm/s, no damage expected.
- >10 mm/s and <20 mm/s – notify CN for assessment.
- 20 mm/s or more – stop work immediately.

Automatic alarms were set to provide notifications if the vibration readings exceeded the limits. Rationale for the values of vibration limits and their applicability to this specific project was not provided or discussed.

4.2 Vibration Monitoring Results, Impact to Bridge.

Figure 6 shows typical results during pile driving – operation of the pile driving hammer is clearly identifiable, with accelerations during pile driving generally within the lower threshold of 10 mm/s. The upper alarm limit of 20 mm/s was rarely met, for durations of less than 30 seconds, for which the impact to the bridge was believed negligible. The time required for vibrations exceeding the upper alarm limit to impose damage to the bridge was not assessed – vibration data was continually monitored, and work was to be stopped immediately if significant exceedance was noted. The judgment of the occasional sub – 30 second event being non-significant was based on the experience of the vibration monitoring consultant. As significant vibration was not experienced, consideration of minimizing vibration from the pile driving hammer was not considered.

Accelerations in the x, y and z direction were comparable, with no distinct trend of higher or lower magnitude in any particular direction. The vibration amplitudes were generally higher for the sensors mounted at the base of the piers during pile driving, and higher at the top sensors during passing of trains. The scope of vibration monitoring was limited to determining whether the vibration limits were met or exceeded; therefore, detailed review of the magnitude of vibrations was not conducted.

5.0 Deflection Monitoring

Monitoring of the deflection of the bridge structure was conducted daily throughout construction to identify and record any permanent movement due to pile driving activities. Monitoring included tilt loggers and surveying of a series of prisms installed on the bridge piers and superstructure.

5.1 Tilt Loggers

DTL202B biaxial tilt loggers with DTLINK wireless connectivity, supplied by RST Instruments, were installed on Piers 6 and 8, and oriented to record tilt in the x and y directions.

The loggers were set to record every 30 minutes. The output of the loggers was the sine of the angle of tilt. Using the height of the loggers above the base of the pier, the lateral deflection at the bearing seat was calculated. Figure 7 shows a typical plot of lateral deflection and temperature vs. time for tilt loggers and survey data.

5.2 Survey of Bridge Deflections

Survey prisms were installed at various locations on the bridge piers and superstructure. The proportionality between the survey measurements and calculated deflections from the tilt logger data was good – prisms installed near the tilt loggers showed similar deflections.

5.3 Observed Deflections

The deflections calculated from tilt logger data and the movement observed by survey monitoring both correlated well with ambient temperature fluctuations, as seen in Figure 7. No significant movement was observed as a direct result of pile driving. Pile driving was continual thoughout most work days for the duration of the project – it is believed that the 30-minute recording interval for the tilt loggers was appropriate to capture potential deflections as a result of pile driving.
Lateral deflections of the piers were limited to about 15 mm, and were believed to be due to fluctuations in temperature, which ranged from around 0 degrees Celsius, down to approximately -30 degrees Celsius throughout the project. Tilt and vibration monitoring was not continued after completion of piling.

6.0 Conclusions

The bearing strata for the piles was very hard glacial till, with laboratory measured undrained shear strength in the range of 700 kPa or more. With relatively hard driving conditions expected, the steel H section HP310x110 piles were driven successfully using a Junttan HHK5A hydraulic hammer. Initially, the boulder lag along the riverbed posed difficulty in initial driving; however, once the boulder lag was removed, piling progressed at a rate of approximately 15 per day, on average. Pile capacities for new piers and existing piers were close to 3,000 kN after seven days.
It is suspected that the in-situ undrained shear strength of the Sutherland till was higher than the laboratory measured results, possibly due to stress release and subsequent small-scale sample disturbance during sampling, as well as scale effects related to fractures and fissures in the till.

No significant impact to the bridge was observed during pile driving. Vibration monitoring showed accelerations that generally remained below 10 mm/s, with occasional short occurrences (less than 30 seconds) above 10 mm/s, and rare occurrences (less than five throughout the project) above 20 mm/s.

No significant permanent deflection of the bridge was observed, with tilt logger data and survey data being proportional, and corresponding well with ambient temperature fluctuations.

7.0 References

Christiansen, E. A. (1968). Pleistocene stratigraphy of the Saskatoon area, Saskatchewan, Canada. Canadian Journal of Earth Sciences, 5(5), 1167-1173.

Christiansen, E. A. (1970). Physical Environment of Saskatoon, Canada.

Tables

Table 1 – Existing Piers Termination Criteria

WEAP			Test Piling Program		
Re-Strike Period (days)	Minimum Embedment Length (m)	Blows/m	Re-Strike Period (days)	Minimum Embedment Length (m)	Blows/m
3	9.1	14	1	9.0	8
6	8.5	13	-	-	-

Table 2 – New Piers Termination Criteria

WEAP			Test Piling Program		
Re-Strike Period (days)	Minimum Embedment Length (m)	Blows/m	Re-Strike Period (days)	Minimum Embedment Length (m)	Blows/m
3	15.5	30	1	11.0	17
6	14.5	27	-	-	-

Figures

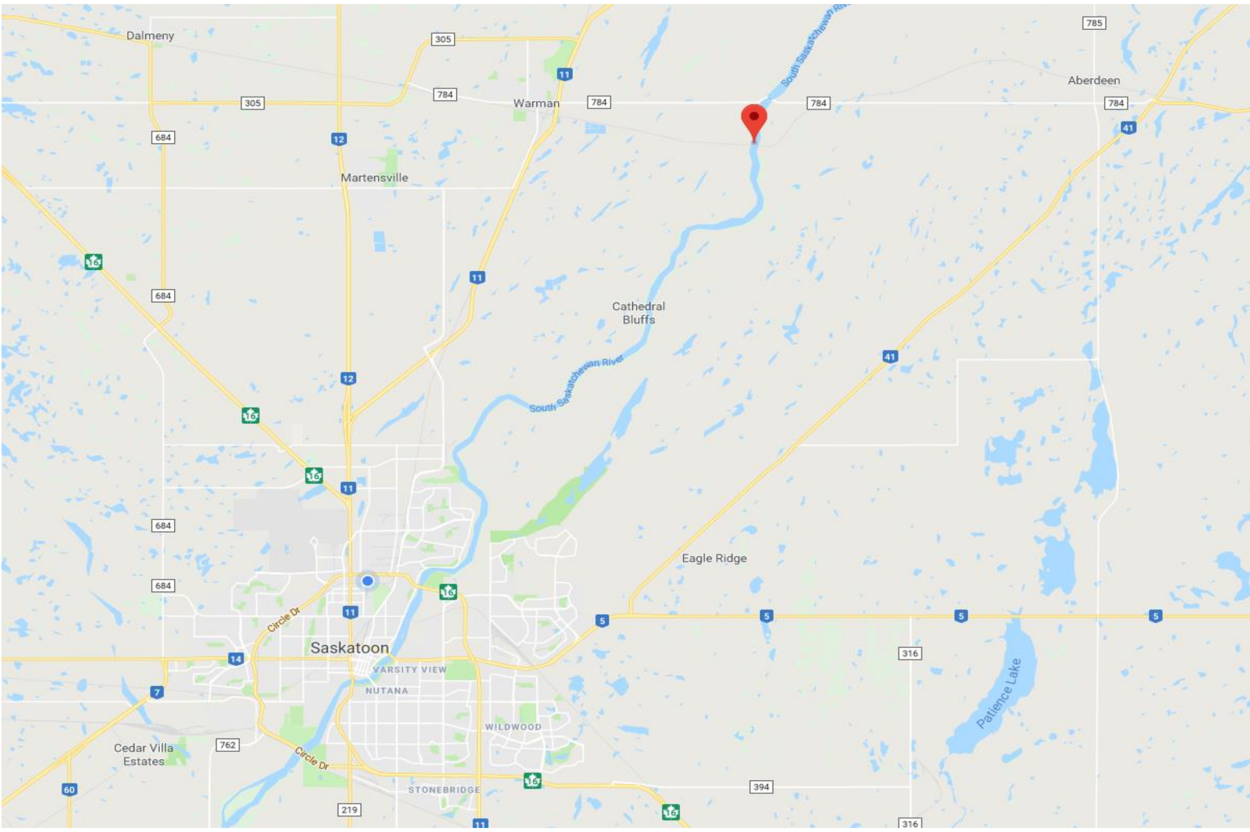

Figure 1 – Project Location

TABLE III. DRIVEN STEEL H-PILE BEARING PRESSURES (WEST APPROACH)

Zone (metres)	*Allowable Compressive Bearing Pressures (kPa)	
	Skin Friction	End Bearing
0 to 2 (or fill depth, whichever is greater)	0	–
2 to 5	35	–
Below 5	50	1,000

*Based on Factor of Safety = 2.5

TABLE IV. DRIVEN STEEL H-PILE BEARING PRESSURES (EAST APPROACH)

Zone (metres)	*Allowable Compressive Bearing Pressures (kPa)	
	Skin Friction	End Bearing
0 to 2 (or fill depth, whichever is greater)	0	–
2 to 6	20	–
6 to 9	30	–
Below 9	50	1,000

*Based on Factor of Safety = 2.5

TABLE V. DRIVEN STEEL H-PILE BEARING PRESSURES (PIERS)

Zone (metres)	*Allowable Compressive Bearing Pressures (kPa)	
	Skin Friction	End Bearing
0 to 2 (Below River Bottom)	0	–
2 to 5	35	–
5 to 16	60	1,200
Below 16	70	1,500

*Based on Factor of Safety = 2.5

Figure 2 – Foundation Design Parameters (taken from geotechnical investigation report "CN Aberdeen Mile 60.8 Bridge repair/replacement NE ¼ 26-38-4 W3M near Warman, Saskatchewan" by P. Machibroda Engineering Ltd. (PMEL File No. 12606))

Figure 3 – Piling at Pier 7

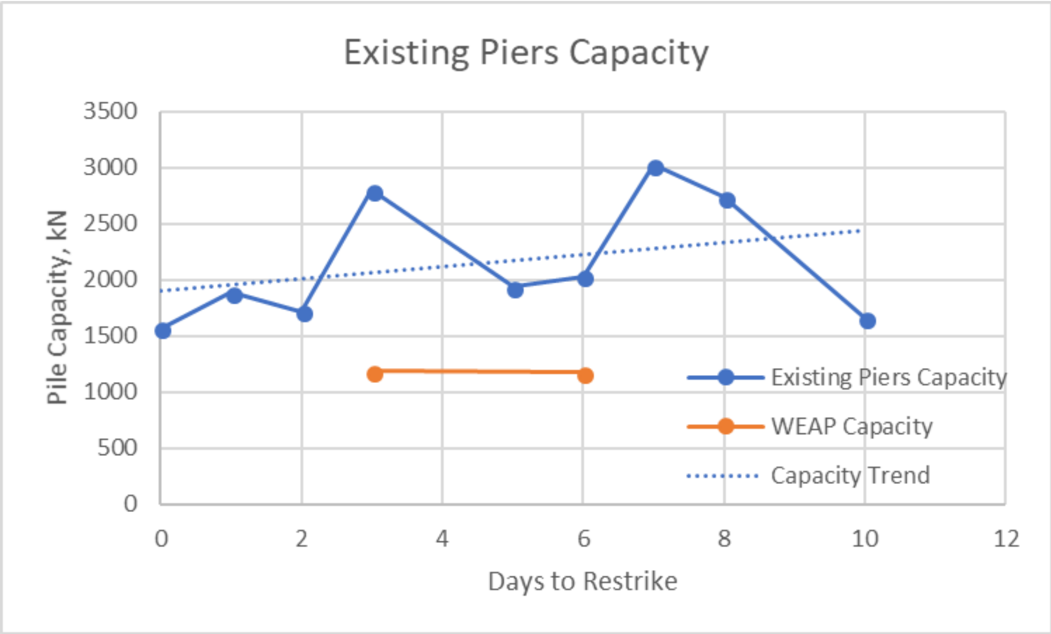

Figure 4 – Pile Capacities – Existing Piers

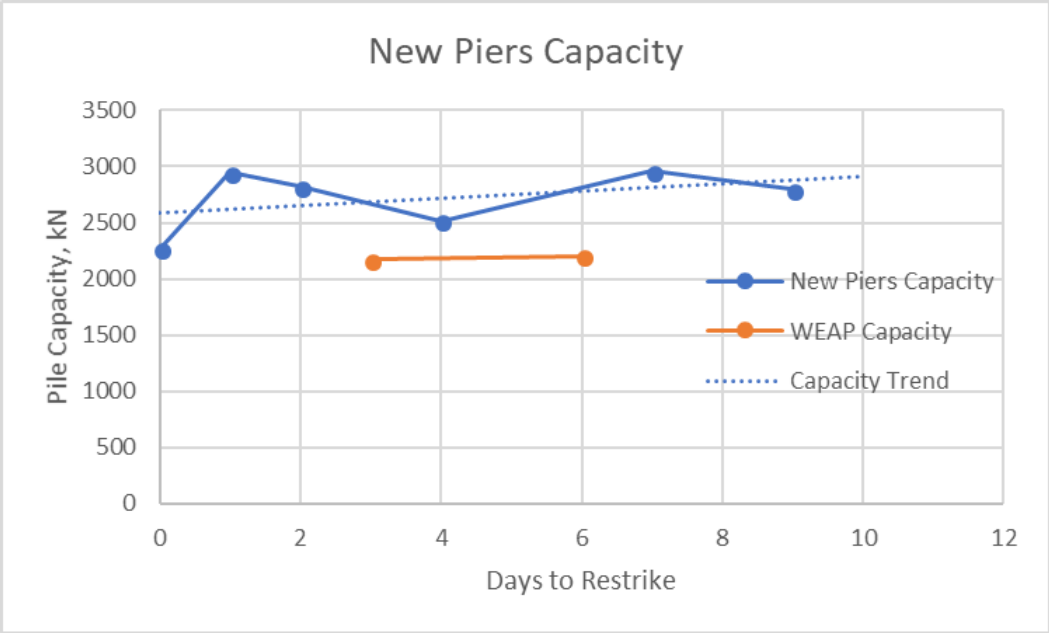

Figure 5 – Pile Capacities – New Piers

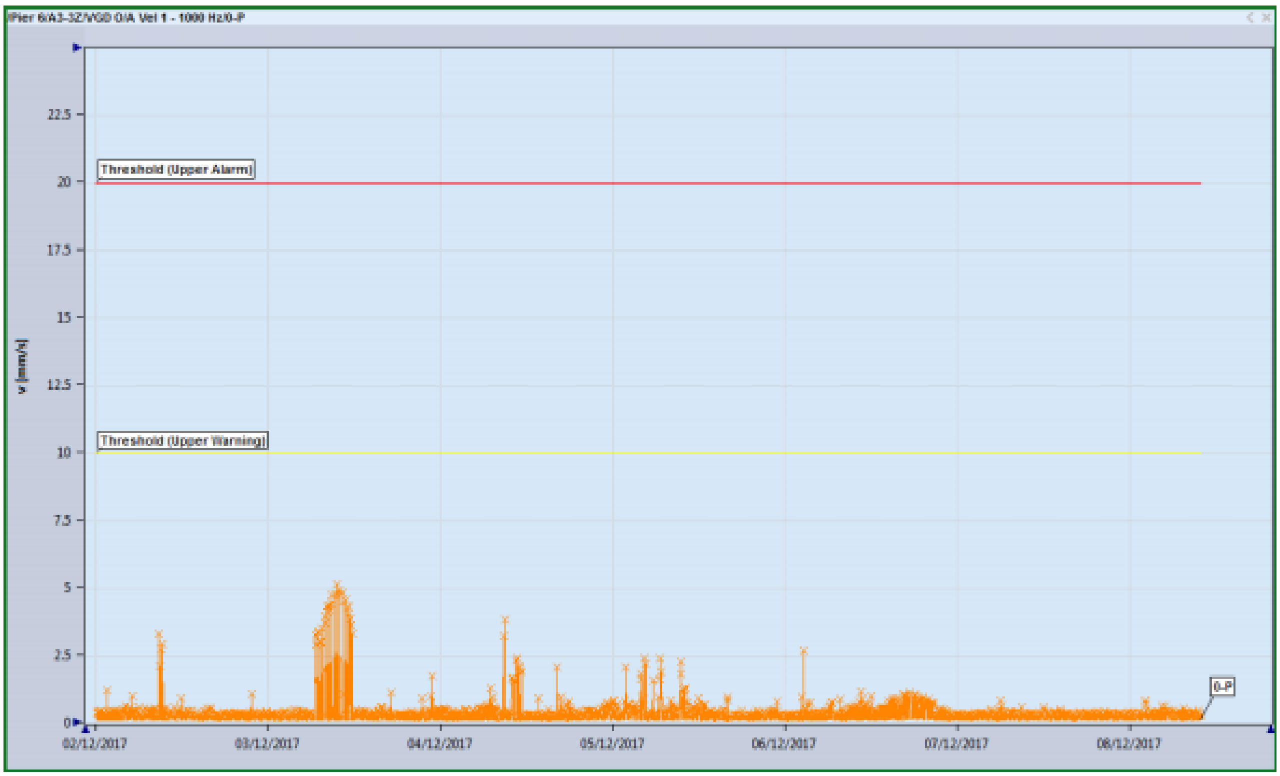

Figure 6 – Example of vibration monitoring, Pier 6

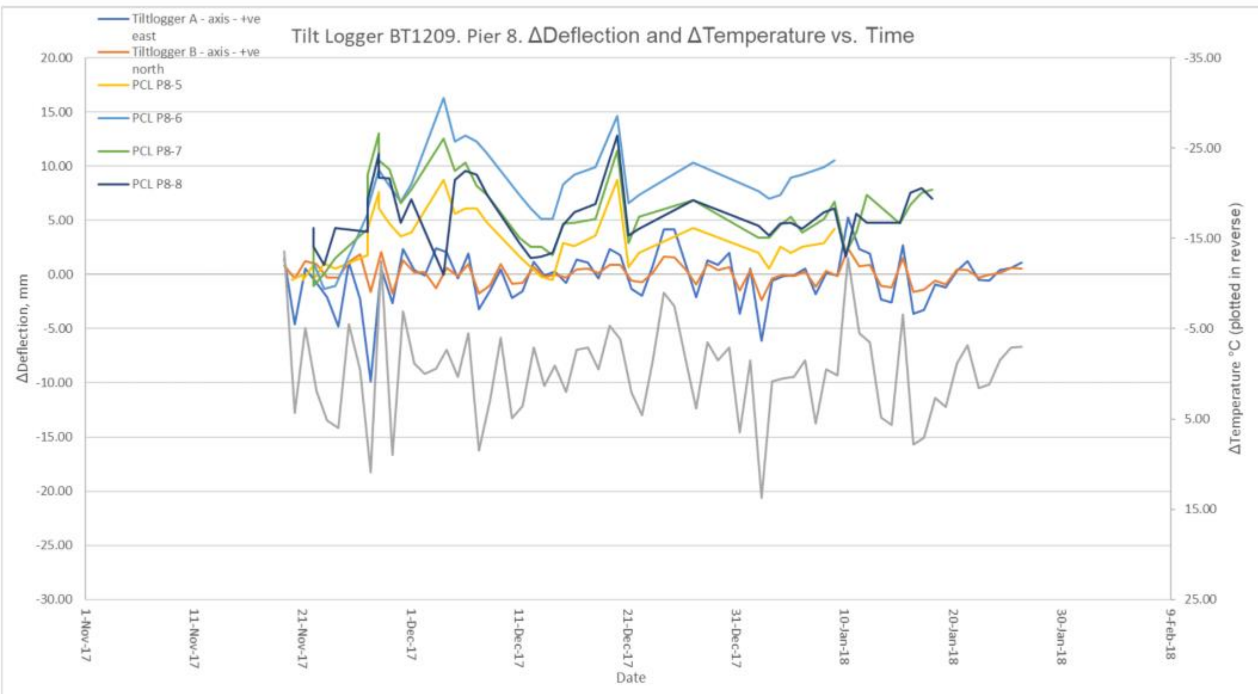

Figure 7 – Tilt Logger, Temperature and Survey Data, Pier 8