18 October 2016

GEOTECHNICAL DATA REPORT - STATIC PILE LOAD TEST

George Massey Tunnel Replacement Project

Submitted to:
BC Ministry of Transportation and Infrastructure
George Massey Tunnel Replacement Project
2030-11662 Steveston Highway
Richmond, BC
V7A 1N6

Report Number: 1533603-027-R-Rev1

Distribution:

1 copy - BC MoTI
1 copy - Golder Associates Ltd.

REPORT

Golder Associates

Executive Summary

Golder Associates Ltd. (Golder) was retained by the British Columbia Ministry of Transportation and Infrastructure (MoTI) to conduct a fully instrumented static pile load test for the George Massey Tunnel Replacement Project at the Test Pile site located adjacent to the existing south portal of the George Massey Tunnel (GMT) in Delta, BC. This report documents the results of the construction and Static Load Test activities carried out by Golder at the Test Pile site between November 2015 and August 2016, as authorized by MoTI.

In summary, the Static Load Test project involved the following activities:

- Mobilization and demobilization to/from the Test Pile site; including construction of access roads, working (crane) pads and laydown areas;
- Installation of vibration sensors, deep settlement gauges, survey hubs, automated survey system, concrete joint extensometers and noise monitoring instrumentation at locations defined by MoTI to monitor and record effects related to pile installation activities;
- Installation of five, 2 m diameter open toe steel pipe piles to elev. -66 m in a pattern meeting the minimum spacing requirements outlined in ASTM 1148M-07. The 25 mm thick reaction piles were located at the outside corners of an approximately square pattern and the 32 mm thick Test Pile was located at the center of the square pattern. The test pile was fully instrumented with 8 levels of strain gauges and two telltale rods that extended from near the pile tip to above the ground surface (about 67 m length);
- Fabrication and assembly of a steel reaction frame that was structurally connected to the reaction piles. The reaction frame was centered symmetrically over the Test Pile and was designed to provide a reaction load to the hydraulic jacks during the Static Load Test;
- Installation and monitoring of geotechnical instrumentation on the Test Pile and reaction piles including vibrating wire strain gauges, telltale rods/housings and LVDT sensors, load cells, liquid level settlement monitoring system and laser tracking survey system. Fabrication and installation of an automated data acquisition system (ADAS) to record and display the instrumentation readings obtained during the Static Load Test;
- Installation of a hydraulic jacking system, including hydraulic pumps/reservoirs, valves, hoses and jacks to apply and control loads during the Static Load Test;
- Completion of three individual phases of the Static Load Test including the initial loading/unloading phase and the second loading/unloading phase on the originally designed Test Pile and the third loading/unloading phase on the modified Test Pile configuration (concrete plug extended to top of Test Pile); and
- Compilation and reporting of Static Load Test information.

Pile installation activities took place between March 12, 2016 and June 7, 2016 and started with installation of the reaction piles and finished with installation of the Test Pile. The construction monitoring results were recorded, compiled and provided to MoTI at the end of each day of pile installation. The construction monitoring results during pile installation are summarized as follows:

- Vibration Monitoring: measured Peak Particle Velocity (PPV) between about 1 mm/s and 27 mm/s at various locations on the existing tunnel structure. In general, the maximum PPV values were recorded during installation of the second segment of each pile using the APE D138-42 diesel hammer;
- Joint Meter Monitoring: measured joint movement between about 1 mm and 5 mm. In general, the maximum displacements recorded by the joint meters did not correspond directly with the pile driving activities and, based on review of the joint monitoring displacement data, it is considered likely that the measured joint meter displacements were more related to external temperature and/or tidal effects in the adjacent Fraser River;
- Deep Settlement Gauge Monitoring: measured vertical (downward) displacement between about 10 mm and 25 mm. The maximum displacements in the deep settlement gauges were generally recorded during installation of reaction piles RP3 and RP4 (the deep settlement gauges adjacent to RP3 and RP4 recorded the highest displacements);
- Survey Monument Monitoring: measured absolute displacement between about 1 mm and 10 mm. Higher displacement values were recorded by the survey instruments during pile installation activities due to vibration of the total stations while measurements were occurring; however, the survey readings generally stabilized to values close to the original readings once pile driving stopped. Overall, the displacements recorded by the survey equipment did not appear to directly correspond to the pile installation activities and, based on review of the survey measurements, it is considered likely that the survey monument displacements were more related to external temperature and/or tidal effects in the adjacent Fraser River;
- Ground Surface Monitoring: measured vertical (downward) displacement between about 30 mm and 460 mm. The highest displacements were typically measured where ground deformation related to liquefaction of the soils adjacent to pile installation occurred; and
- Noise Monitoring: Lmax measurements between about 99 and 121 dBA, Leq measurements between about 73 dBA and 97 dBA, and Lpeak measurements between 122 dBA and 134 dBA. The largest increases in noise levels during pile driving using the diesel hammers were generally recorded at the measuring locations directly adjacent to pile driving activities.

No visible damage due to construction activities was observed to the Massey Tunnel structural elements based on comparison of the pre and post condition surveys that were carried out. Some ground surface subsidence was observed between the Massey Tunnel and the Test Pile site during pile installation when the vibratory hammer was used.

Final installation of the Test Pile (including cleanout, installation of a 20 m long concrete plug, installation of the load plates, load cells and hydraulic jacks) occurred on July 6, 2016 and was immediately followed by test frame assembly and final installation of geotechnical instrumentation. Final assembly of the test frame and installation of geotechnical instrumentation was completed on August 17, 2016.

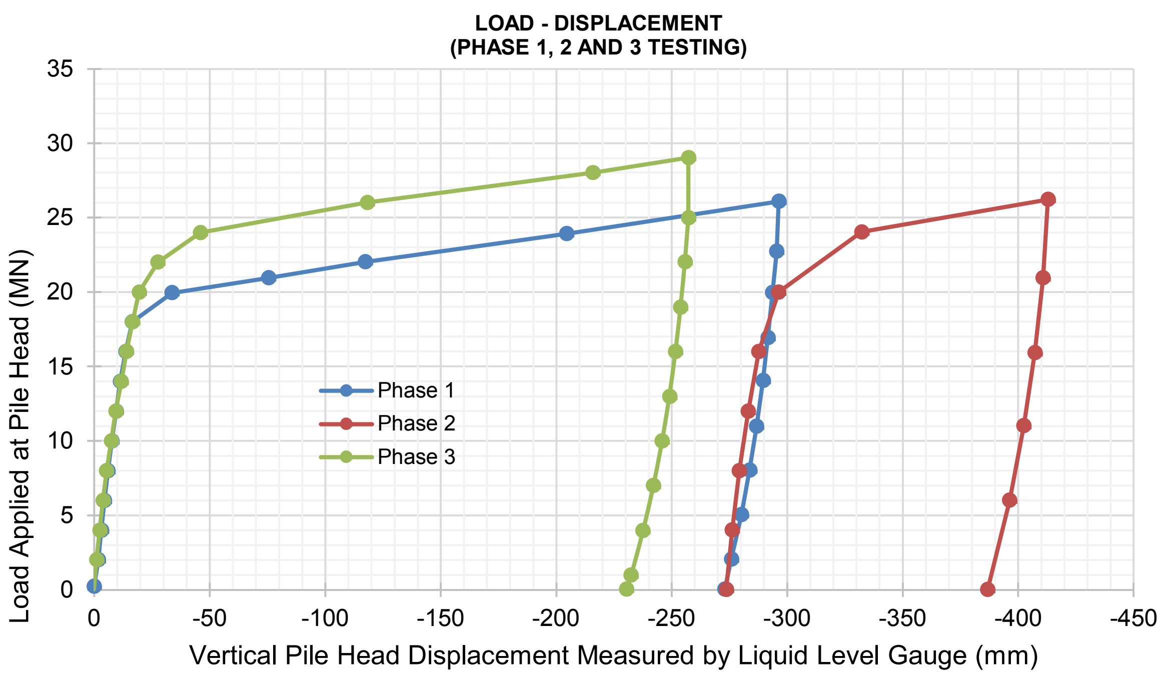
The Static Load Test was carried out in three separate loading/unloading phases including:

- An initial loading and unloading phase on August 18, 2016 carried out on the original Test Pile configuration to 26.2 MN maximum applied load and 296 mm vertical pile head displacement;

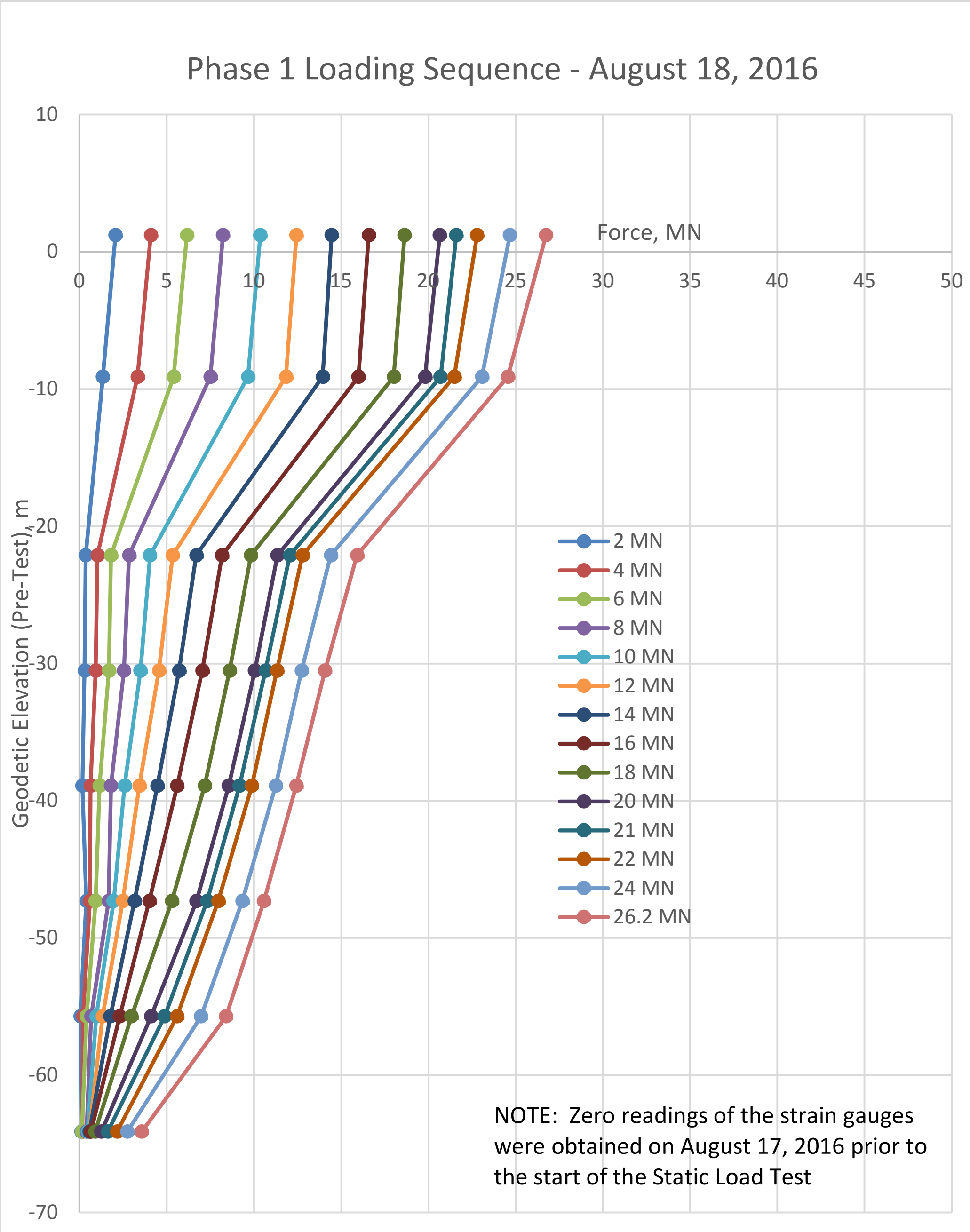
- A second loading and unloading phase on August 19, 2016 carried out on the original Test Pile configuration to 26.7 MN maximum applied load and 413 mm vertical pile head displacement (total combined displacement including the first loading and unloading phase); and
- A third loading and unloading phase on August 31, 2016 carried out on a modified configuration of the Test Pile where the concrete plug was extended to the top of the Test Pile (the bottom of the lower load plate). The third phase of the Static Load Test was taken to 29.1 MN maximum applied load and 257 mm vertical pile head displacement (644 mm combined vertical pile head displacement for all three phases of the test).

The three phases of the Static Load Test were completed successfully.

A summary chart displaying the load vs. displacement curves obtained during the three phases of the Static Load Test is presented below:



The instrumentation data obtained during the three phases of the Static Load Test was compiled and was summarized graphically and in tabular format in the Geotechnical Data Report (Report). The strain gauge data obtained from the Test Pile during the Static Load Test has was assessed to estimate the load distribution along the pile shaft prior to the Static Load Test and for each phase of loading and unloading during the Static Load Test. The load distribution estimates are presented graphically and in tabular format in the Report. A sample plot of load distribution along the pile shaft for the first loading sequence is presented below.



The raw data set obtained from the all of the geotechnical instrumentation has been provided to MoTI in electronic format. The instrumentation data presented in the Report summarizes the MoTI specified data sets (a minimum contract requirement) and does not include all of the data provided in the electronic files.

Table of Contents

TABLES

FIGURES

APPENDICES

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by the British Columbia Ministry of Transportation and Infrastructure (MoTI) to conduct a fully instrumented static pile load test for the George Massey Tunnel Replacement Project at the Test Pile site in Delta, BC. This report documents the results of the construction and static pile load test activities carried out by Golder at the Test Pile site, located adjacent to the existing south portal of the George Massey Tunnel, between November 2015 and August 2016, as authorized by MoTI. This report addresses the work carried out for the Static Load Pile Test program as defined by the RFP/contract and specifically does not include site characterization, interpretation of the pile test results (except as defined herein), nor engineering assessment or recommendations.

Engineering comments and recommendations regarding the geotechnical aspects of the George Massey Tunnel Replacement Project, including interpretation of the results of the static pile load test are beyond the scope of our assignment and are not addressed herein. MoTI specifically defined the scope of the static pile load test and construction monitoring reporting requirements in Consulting Services Contract 159CS0008. Golder was not involved in planning or design of the static load test and was not aware of any of the project design requirements, including pile design and capacity requirements, at the time of the construction and pile load test work. As such, Golder cannot guarantee or warranty that the geotechnical information obtained is sufficient to fully satisfy the overall project objectives or requirements.

This report should be read in conjunction with the **"Important Information and Limitations of This Report"** which is attached following the text of this report. The reader's attention is specifically drawn to this information as it is essential for the proper use and interpretation of this report.

2.0 PROJECT DESCRIPTION AND LOCATION

As part the of data collection, preliminary planning and design phase(s) of the George Massey Tunnel Replacement Project (GMTRP), MoTI required a geotechnical investigation consisting of a full scale, fully-instrumented, static pile load test to:

- Determine the effects of pile installation on the existing soil formations;
- Determine the effects of pile installation on the existing Massey Tunnel structural elements; and
- Evaluate the suitability of steel pipe pile foundation for the proposed replacement structure.

MoTI retained Golder to carry out the pile test work to obtain specific measurement data on the effects of pile installation to sensitive adjacent structures (the existing Massey Tunnel and south approach) and specific measurement data on pile load-deflection response and pile load transfer during execution of the full-scale static pile load test. The objective of the work was to acquire and provide to MoTI the data collected during construction and during the Static Load Test thereby allowing assessment of the above items by others.

The location of the Test Site is directly adjacent to the south portal of the Massey Tunnel on Deas Island in Delta, BC, as presented in Figures 1 and 2. It was a requirement that the existing Massey Tunnel remain fully operational during all of the site work and specialized monitoring and construction procedures were required to minimize potential effects to Highway 99 traffic and the Massey Tunnel and south approach structures. MoTI provided a clear outline on vibration and deformation thresholds and construction procedures that needed to be implemented to mitigate effects due to pile installation should they be required.

The general scope of work for the assignment included the following:

- Site Preparation including working/crane pad construction, site security setup and construction trailer setup;
- Design and fabrication of the pile test loading frame and jack arrangement meeting the requirements of the RFP/contract Special Provisions and ASTM D1143M-07;
- Supply and installation of specific instrumentation to monitor vibration and deformation on the existing Massey Tunnel and south approach structures as well as noise in the general vicinity of the work during pile installation;
- Installation of grouting pipes at locations specified by the Ministry as a potential remedial measure of settlement in the area adjacent to the Massey Tunnel south approach walls;
- Supply and installation of five, large diameter, steel pipe piles as specified in the contract documents;
- Supply and installation of the required test pile instrumentation including strain gauges, load cells, liquid level gauges, telltales/LVDTs, and survey instruments as specified in the RFP/contract documents;
- Supply and installation of an Automated Data Acquisition System (ADAS) to acquire and store the instrumentation data obtained during the Static Load Test;
- Supply and operation of all equipment necessary to install the test and reaction piles, the load frame, jacking equipment and instrumentation to carry out the Static Load Test;
- Provide daily reports on adjacent structure inspection/monitoring, including vibration, deformation and noise, as well as daily reports presenting the Static Load Test data; and
- Provide a final geotechnical data report deliverable in a format consistent the requirements set out in ASTM D1143M-07 including the complete data set obtained during the work.

The entire Static Load Test was designed, set-up and executed in general accordance with the requirements set out in the RFP/contract documents and ASTM D1147M-07 – "Standard Test Methods for Deep Foundations Under Static Axial Compressive Load". Any deviation from the ASTM methodology is documented in Section 12 below.

3.0 TEST SITE

The Test Pile site is located immediately east of the south portal to the existing George Massey Tunnel in a grassy field area as shown in Figure 2. The existing George Massey Tunnel is a four-lane structure, approximately 1.5 km in length that extends beneath the south arm of the Fraser River from south Richmond to Deas Island. A short four-lane bridge connects the south end of Deas Island to the Delta mainland.

Drainage ditching and flood-protection earthen embankments have been constructed along the banks of the Fraser River, and encompass the north and south entrances to the tunnel. Based on verbal discussions with MoTI, these embankments are understood to have been constructed of Fraser River dredge sand and possible coarse riprap. To the east of the tunnel's south entrance, and generally through the western portion of the Test Pile site, a drainage ditch runs north-south for approximately 1 km. A paved maintenance/access road loops around the south portal of the tunnel and Test Pile site allowing bypass access for maintenance and emergency vehicles between the northbound and southbound travel lanes on Highway 99. It is believed, based on review of historical information, that the main excavation for the Massey Tunnel entry/exit approach ramps extended out to the paved maintenance/access roads and that the areas contained within the paved maintenance/access road were backfilled up to the current ground elevation following construction of the approach ramps.

Current land-use adjacent to the Test Pile site consists of recreational and undeveloped land on Deas Island. In general, the existing site topography at the Test Pile site is relatively flat and ranges in elevation from approximately +1 m to +4 m geodetic, with the higher elevations generally related to the previous flood-protection (diking) works.

3.1 Construction Site

The designated construction work site encompassed an approximate 80 m by 150 m area within the existing grassy field area referenced above, as shown in Figure 2. The construction site generally included the following facilities:

- Security fencing around the perimeter of the work site including two entry/exit gates to/from the existing maintenance/access roads;
- A 5 m to 8 m wide gravel haul road between the entry/exit gates to facilitate delivery and construction equipment traffic;
- An approximate 20 m by 40 m, 0.7 m to 1.0 m thick, gravel pad at the test pile/frame location to facilitate crane traffic during pile installation and erection of the load frame;
- An approximate 25 m by 40 m, 0.3 m thick, rectangular gravel pad south of the gravel crane pad to be utilized as a laydown area;
- Up to six construction trailers located at various locations on the site including two large office trailers, one small office trailer, one small first aid trailer and one small equipment trailer at the south end of the fenced construction site and one small instrumentation trailer at the north end of the site; and
- Electrical and water supply was provided by Mainroad Construction from existing tunnel facilities.

Access to portions of the existing ventilation tunnel and ventilation building were made available to Golder throughout the project for inspection and construction monitoring purposes.

4.0 PROJECT TEAM

4.1 Owner

The owner for the project is the British Columbia Ministry of Transportation and Infrastructure (MoTI). The project is managed by a team specifically arranged by MoTI for the George Massey Tunnel Replacement Project. The project team's address is:

2030 – 11662 Steveston Highway
Richmond, BC
V7A 1N6

4.2 General Contractor

The General Contractor for the project is Golder Associates Ltd. (Golder) under MoTI contract number 159CS0008. The contract is managed from Golder's Vancouver office at:

200 – 2920 Virtual Way
Vancouver, BC
V5M 0C4

Several sub-contractors were retained to carry out the work. The names and locations of the key sub-contractors are provided in the section below.

4.2.1 Geotechnical Engineer

The geotechnical engineering services for the General Contractor were provided by Golder. The Lead Geotechnical Engineer for the project was Mr. Pierce Bakker, PEng, Associate. Technical oversight and review was provided by Mr. Yogi Yogendrakumar, Ph.D., PEng, Principal. Both engineers are based in Golder's Vancouver office.

4.2.2 Structural Engineer

The structural engineering services for the pile test load frame and pile jacking design were provided by All-Span Engineering and Construction Ltd. (All-Span) based from their office at:

201 – 7198 Vantage Way
Delta, BC
V4G 1K7

The Lead Structural Engineer for the test frame and jacking design was Mr. Barry Gerbracht, PEng.

It is noted that the structural engineering design for the test pile and reaction piles was provided by MoTI prior to project award.

4.2.3 Pile Sub-Contractor

Fraser River Pile and Dredging Ltd. (FRPD) was retained to install each of the reaction and test piles, install the protection for the sub-surface geotechnical instrumentation that was installed on the test pile, cleanout of upper 20 m of the Test Pile, and placement of the concrete plug. FRPD's headquarters are located at:

1830 River Drive
New Westminster, BC
V3M 2A8

4.2.4 Load Frame Fabricator

Macform Construction Group Ltd. (Macform) was retained to fabricate and install the pile test load frame. Macform's headquarters are located at:

10136 201st Street
Langley, BC
V1M 0B1

4.2.5 Geotechnical Instrumentation Supplier

The geotechnical instrumentation and Automated Data Acquisition System equipment for the project was supplied by RST Instruments Ltd. (RST). Their headquarters are located at:

11545 Kingston Street
Maple Ridge, BC
V2X 0Z5

4.2.6 Other Team Members

Other project sub-contractors/suppliers of particular note include:

- Mainroad Contracting Ltd. of Richmond, BC (traffic management, water and electrical supply, access to Massey Tunnel);
- Underhill Geomatics Ltd. of Burnaby, BC (site layout, construction survey monitoring, as-constructed survey);
- Hymac Industries Ltd. of Vancouver, BC (jacking supply and operation);
- Pipe and Piling Supplies Ltd. of Port Coquitlam, BC (steel pipe pile supply); and
- Acuren Group Inc. of Richmond, BC (steel pipe inspection and testing).

5.0 GEOTECHNICAL INVESTIGATION AND SUBSURFACE INFORMATION

Golder did not obtain any specific subsurface information as part of the Static Pile Load Test contract. All geotechnical investigation and subsurface information was provided by MoTI based on information obtained during previous investigations that were carried out. Specific details about the information supplied by MoTI are provided below.

5.1 MoTI Supplied Information

5.1.1 Test Hole Types

MoTI supplied a total of three electronic cone penetration test (CPT) plots and one drilled borehole summary log in the Request for Proposal and contract documentation. These included:

- CPT15-06, CPT15-07 and CPT15-08 put down to 100 m depth below the existing ground surface (each); and
- BH15-01 drilled rotary borehole put down to 201.8 m depth below the existing ground surface.

The specific locations of the test holes listed above are presented in the geotechnical data report presented in Appendix A. In general, the test holes were located in close vicinity to the Test Site area.

5.1.2 In-Situ and Laboratory Test Results

The results of all in-situ and laboratory results obtained during the previous geotechnical investigation are contained in the geotechnical data report presented in Appendix A. In addition, a summary of the laboratory gradation and index property results for selected samples is presented on the BH15-01 summary log presented in geotechnical data report. The in-situ CPT measurements are displayed on the CPT plots in the geotechnical data report presented in Appendix A.

6.0 PROJECT WORK PLANS

As required by the RFP/contract, Golder submitted various work plans to MoTI prior to and during the construction (piling and load frame construction) work. These work plan submittals included:

- Mobilization and Demobilization Work Plan;
- Adjacent Structures Monitoring Work Plan;
- Instrumentation and Data Collection Work Plan;
- Pile Installation Work Plan;
- Settlement and Joint Remediation Work Plan; and
- Static Load Test Work Plan.

A copy of each of the above final work plan submittals is presented in Appendix B.

7.0 MONITORING SYSTEMS/INSTRUMENTATION

7.1 Monitoring Requirements

Due to the close proximity of the pile installation activities and associated ground disturbance to the existing Massey Tunnel and nearby populated areas, a site-specific construction monitoring program was required in the RFP/contract. The purpose(s) of the monitoring program was to:

- Assess the pre-construction conditions of the adjacent structures within 100 meters of the proposed pile installation activities;
- Actively monitor ground settlement, vibration and deformation of the South Approach, Massey Tunnel and adjacent ground during pile installation activities;
- Monitor construction-related atmospheric noise on and around Deas Island during pile installation activities;
- Immediately determine if pile driving activities impact the adjacent structures beyond the threshold limits established by the Ministry;
- Establish a clear means of communication with the Ministry should any damage to the adjacent structures occur and/or should any of the established thresholds for vibration and deformation be exceeded during pile installation;
- Record the condition of the adjacent structures daily during pile installation; and
- Assess the post-construction condition of the adjacent structures following completion of pile installation activities, including monitoring of settlement and deformation monuments for a period of five days following the installation of the piles.

A detailed description of each of the monitoring components carried out during construction is provided below.

7.2 Construction Inspections

A visual condition assessment of all of the adjacent structures within 100 m of the pile installation area, including the ventilation building, south approach walls and southern most tunnel segment was carried out prior to construction (i.e. pile driving activities). The initial assessment included development of an inventory and photographing visible areas of existing disrepair. The initial assessment was conducted by a qualified structural engineer from All-Span in the presence of the Ministry Representative. A copy of the pre-construction condition report summarizing the results of the inspection prepared by All-Span is included in Appendix C.

A brief visual inspection of the inventoried disrepair areas was conducted each day that pile installation activities occurred and the results presented in the daily monitoring report(s) provided to MoTI at the end of each day of pile installation.

After pile installation was fully completed, a final visual inspection of the ventilation building, tunnel approach walls and southern most tunnel segment was carried out by All-Span in the presence of the Ministry Representative.

Copies of the daily and post-construction condition reports are also included in Appendix C.

7.3 Automated Survey System

A series of survey monuments were installed at the locations specified by MoTI in the RFP/contract documents including areas on the top of the tunnel approach walls, interior maintenance tunnel wall, ventilation building, deep settlement gauges and adjacent ground surface monuments. In addition, a series of survey control monuments were also installed in areas located outside of the construction site. The survey monuments (Leica GMP104 mini-prisms) were securely fastened to either the existing concrete, the side of deep settlement gauges and top of the ground surface hubs (pins) at the general locations identified in Figure 3. The monuments were surveyed during pile driving activities using a Robotic Total Station Monitoring (RTSM, Lieca Nova TM50/MS50 Monitoring Total Stations) survey system that was configured to optimize the line of sight between the monuments, RTSMs, and control points as well as meet the minimum 2 minute reading interval requirement specified in the contract. Four of the RTSMs were installed on concrete block pedestals surrounding the construction site and one RTSM was mounted to the maintenance tunnel wall at the locations shown in Figure 3.

The RTSMs were programmed to automatically survey the monuments to an accuracy of approximately 5 mm and were connected to a Leica GeoMos ADAS system for real-time reporting and recording of the measured absolute survey coordinates as they became available. The survey ADAS system was located in the instrumentation trailer (Seacan) and was configured to provide alarm warnings if/when the minimum ground displacement threshold of 25 mm in any direction from the baseline measurement was exceeded. The ADAS system was monitored remotely by Underhill for system maintenance and quality control purposes while it was set up.

To establish the baseline set of coordinate values for all the monitoring and control points, a redundant and closed survey control network was observed at the surface level and down into the ventilation tunnel. The tunnel control traverse was double run, both in and out of the tunnel, for a closed network configuration. All available site lines were measured in the survey network and a three dimensional least squares analysis was performed on the primary network. The final adjusted coordinates were derived from this primary control. The coordinate system for the project was based on the local ground level, with reference to the Zone 10 UTM CSRS coordinate system. The recorded elevations are geodetic (CVD28GVRD) based on the two first order Federal benchmarks 77C029 and 67C147, located near the south entrance of the tunnel.

The data recorded by the Geomos ADAS was downloaded and provided to MoTI at the end of each day of pile installation, or occasionally the day following.

7.4 Joint Meters

A total of five crack/joint meters (RST Instruments VWCM1000) were installed on the eastern wall of the exposed tunnel approach structure at the locations shown on Figure 3. The purpose of the joint meter installations was to directly monitor relative movement between the six adjacent segments of the south approach structure that adjoin to the Ventilation Building. Each end of the joint meters were securely fastened (doweled) into the top of the concrete approach wall on either side of the designated joint and adjusted so that the meters could detect both compression and elongation (100 mm total movement range, 50 mm movement in either direction). The joint meters were connected to an Automated Data Acquisition System (ADAS) located in the instrumentation trailer (Seacan) by cables.

The data recorded by the ADAS was downloaded and provided to MoTI at the end of each day of pile installation, or occasionally the day following.

7.5 Deep Settlement Gauges

A total of five deep settlement gauges were installed approximately 8 m east of the eastern wall of the exposed tunnel approach structure at the locations shown on Figure 3. The purpose of the deep settlement gauge installations was to directly monitor below-ground settlement at/near the approach wall footing elevation. The deep settlement gauges were installed by Sea to Sky Drilling Ltd. using 115 mm diameter (HW) drill casing and consisted of:

- 25 mm diameter hollow steel pipe section with a 75 mm diameter steel plate welded to the bottom; and
- 75 mm diameter Schedule 40 PVC pipe surrounding the steel pipe section to isolate the steel pipe from the surrounding soils.

The deep settlement gauges were installed with the steel plate located at approximately the same elevation as the approach wall footing immediately adjacent to each deep settlement gauge with the steel rod extending some 1.2 m to 1.5 m above the ground surface. The surrounding PVC pipe acted as an open sleeve for the steel rod to slide freely and extended from approximately 0.2 m above the steel plate up to slightly above the existing ground surface. The installation elevations are provided in Table 1 below and the as-constructed records are included in Appendix D.

Table 1: As-Constructed Deep Settlement Gauge Details

	DSG1	DSG2	DSG3	DSG4	DSG5
Anchor Plate Elevation (m)	-13.5	-13.2	-12.9	-12.3	-11.8
Steel Pipe Riser Pipe Length (m)	16.8	16.8	15.2	15.2	15.2
Steel Pipe Riser Cut-Off Elevation (m)	3.3	3.6	2.3	2.9	3.4

As discussed above, survey monuments (Leica GMP104 mini-prisms) were installed onto the side of the above-ground portion of the steel rods (on each individual deep settlement gauge) for inclusion into the overall automated survey network (a discussion of the automated survey network is provide in Section 6.3 above).

The data recorded by the Geomos ADAS was downloaded and provided to MoTI at the end of each day of pile driving, or occasionally the day following.

7.6 Vibration Sensors

A total of six vibration sensors (geophones) were installed at locations specified by MoTI, including locations on the existing east and west approach walls, ventilation building and southern tunnel segment. The vibration sensors were typically mounted and securely fastened using treaded fasteners that were drilled and epoxied into concrete, except for one geophone that was loosely placed on an existing electrical transformer in the ventilation building (a sand bag weight was used to anchor the geophone). The geophones were connected by cables to Instantel Pro 6 and Pro 4 monitoring/recording devices that were directly compatible with the geophones. During pile driving activities, the Instantel Pro 6 and Pro 4 devices recorded the peak particle velocities (PPVs) in three dimensions every two minutes and were configured such that alarm warnings were triggered if the initial threshold PPV of 15 mm/s was exceeded (peak vector sum). The threshold limit was eventually increased to 30 mm/s at the direction of MoTI and based on review of the vibration monitoring impacts observed during the early stages of pile driving. The vibration monitoring was continuous during pile driving and the peak values monitored every two minutes was recorded. The Instantel readout units were located in the instrumentation trailer, and the real-time data was visually displayed on these readout units.

The data recorded by the recording devices was downloaded and provided to MoTI at the end of each day of pile driving, or immediately the day following.

7.7 Ground Surface Monitoring

The ground surface elevation within and immediately adjacent to the pile driving area was surveyed by Golder on days that pile driving work occurred. The survey work was carried out using a laser level, and specifically included the following areas:

- Areas adjacent to each of the five deep settlement gauges; and
- Three straight lines extending outward and perpendicular to the south approach wall towards reaction pile RP1 (southeast corner of pile test frame), reaction pile RP2 (north east corner of pile test frame) and the test pile TP1 (center of test frame). The ground surface was generally surveyed at 3.0 m intervals along each of the three lines extending to at least 6.0 m east of RP1 and RP2 (the eastern edge of the test frame). The ground surface survey was carried out to a precision of about ±25 mm.

Due to the ongoing construction activity within the pile installation area that resulted in random grading changes and limited access to the survey points, modifications to the ground surface monitoring locations were made to generally exclude ground surface monitoring within the pile driving area. These modifications were agreed to with MoTI as the survey data obtained within the pile driving area was inconsistent and of minimal use.

Elevation measurements were made on the same vertical reference datum as the other survey monitoring. The surface monitoring data was provided to MoTI on a daily basis. The locations used in the ground surface monitoring program are shown in Figure 3.

7.8 Grout Tubes for Settlement Remediation

Four steel, multiple pass sleeve port pipes (or tube-a-manchettes) were installed along the outside edge of the tunnel approach wall footing at the locations shown in Figure 3. The tube-a-manchettes were installed between each of the five deep settlement gauges such that the lowest grout ports were positioned approximately 1.0 m below the base of the approach footing. The ports on the tube-a-manchettes were spaced at 0.4 m intervals along the length of the tubes, with 4 ports at each interval, located at 90 degrees to each other. The tube-a-manchettes were installed by Sea to Sky Drilling Ltd using 115 mm diameter (HW) drill casing. The installation elevations are provided in Table 2 below and the as-constructed records are included in Appendix D.

Table 2: Grout Tube Installation Details

	PGH1	PGH2	PGH3	PGH4
Bottom Tip Elevation (m)	-13.6	-13.2	-12.8	-12.4
Tube-A-Manchette Length (m)	16.8	16.8	15.2	15.2
Cut-Off Elevation (m)	3.2	3.6	2.4	2.8

The purpose of the grout tube installation was to provide an immediate means to mitigate against potential ground subsidence around and beneath the tunnel approach wall footings. Since the measured ground subsidence adjacent to the footings (measured by the deep settlement gauges) did not exceed 25 mm, pressure grouting was not requested by MoTI and the tube-a-manchettes were not utilized.

7.9 Noise Monitoring

Portable noise meters (eight Casella 246 and two Casella 620 noise meters) were installed on and around Deas Island at the ten remote locations shown on Figure 4. The noise meters were installed on existing structures or onto existing features (strapped to utility poles, trees, or the like), out of practical reach of the public. The noise meters were configured to record Lmax and Leq values in A-weighted decibels (dBA) during pile driving activities. At two designated locations, including PLT1 and Site 15, the meters were required to record peak noise levels (Lpeak) in A-weighted decibels. The noise meters were set up to record at the start of each day of pile driving and the data recorded was downloaded and provided to MoTI at the end of each day of pile driving, or immediately the day following.

Photographs of each of the noise monitoring locations/installations are presented in Appendix S.

8.0 PILE INSTALLATION

As specified in the RFP/contract, four 2000 mm diameter by 25 mm thick steel reaction piles and one 2000 mm diameter by 32 mm thick steel test pile were installed at the locations indicated in Figure 5. The actual pile installation activities occurred between March 12, 2016 and May 31, 2016, with test pile cleanout and final cut-off occurring shortly thereafter.

Detailed descriptions of the test pile and reaction pile installations are provided in the following sections below.

8.1 Reaction Piles

8.1.1 Pile Type and Size

The reaction piles were comprised of open ended structural steel pipe segments with nominal dimensions of 2000 mm by 25 mm, welded at designated splice locations. The steel sections were approximately 11.9 m in length (except for single, approximately 3.5 m long sections on each reaction pile) and contained both longitudinal and transverse production welds. The steel pipe pile sections all met ASTM A252-10 Grade 3 specifications upon arrival to the site. A total of twenty-four, 11.9 m long sections were delivered to the site along with four shorter 3.5 m long sections (each reaction pile had approximately 76 m approximate steel length prior to pile driving). The actual installed lengths of the reaction piles varied depending on the amount of pile cut off for fresh heading purposes at each splice location. More detail on the actual installed pile lengths is presented in Table 6 below.

The average dimensions of the reaction piles, based on measurement taken during QC inspection and instrumentation layout were as follows:

- Outside Diameter: 2001 mm;
- Thickness: 24.3 mm; and
- Cross Sectional Area: 150900 mm^2.

These dimensional properties have been utilized in assessing the pile loading prior to, during and after the pile load test.

8.1.2 Identification

For reporting purposes, the reaction piles have been designated as follows in this report:

- RP1 (located at south-east corner of test frame);
- RP2 (located at north-east corner of test frame);
- RP3 (located at south-west corner of test frame); and
- RP4 (located at north-west corner of test frame).

In the quality inspection reporting performed by external parties, and on the Mill Certificates, the reaction piles have been included in Heat Numbers 2520208648, 2520208649, and 2520208662.

8.1.3 Location

As mention above, the reaction piles are located at the corners of the test frame/reaction pile assembly as shown in Figure 5. The as-constructed, surveyed (UTM, NAD 83), centroid of each of the reaction piles are presented in Table 3 below:

Table 3: Reaction Pile As-Constructed Coordinates (Surveyed)

Pile	Northing	Easting
RP1	5440739.543	494743.702
RP2	5440753.390	494732.506
RP3	5440729.526	494731.316
RP4	5440743.550	494720.052

8.1.4 Production Dates/Shipping Dates/Mill Certificates

The reaction piles were manufactured in TianJin, China between October 7, 2015 and November 19, 2015 and were shipped to Port Coquitlam, BC on November 9 and 19, 2015 in two separate shipments.

The Mill Certificates for the reaction piles are presented in Appendix E (reference Heat Numbers 2520208648, 2520208649, and 252020662).

8.1.5 Quality Testing/Inspection Results

Mill Certificates and quality inspection reports were provided for the reaction piles by the pile manufacturer. The Mill Certificates provided by the pile manufacturer indicate that the chemical composition, ultimate tensile strength of the steel and welds, yield strength and dimensional properties all meet ASTM A252, Grade 3 requirements. Further, the quality inspection reports prepared by X&C Engineering in China indicated that the dimensional properties, including diameter, ellipticity and straightness, were all acceptable. Visual inspection and random ultrasonic testing was conducted on the productions welds, where defects were noted, they were corrected and re-inspected. There were no non-conformances noted by X&C Engineering in the final test pile product.

Copies of the relevant Mill Certificates and quality inspection reports are presented in Appendix E (reference Heat Number 2520208648, 2520208649, and 252020662).

In addition to the above quality documentation, Golder commissioned quality testing on the reaction piles (and test pile) following arrival of the steel pipe in Port Coquitlam, BC. The quality testing included chemical composition testing, yield and ultimate tensile testing on coupons obtained from selected areas of the steel pipe and stress-strain testing of steel coupons obtained from selected portions of production welds. All of the additional chemical tests demonstrated compliance with CSA Z662-2015 and Z245.1-2014, and all of the additional yield and tensile tests demonstrated compliance with ASTM A252, Grade 3 requirements.

The additional quality inspection reports, including chemical testing, dimensional properties confirmation, coupon testing and ultrasonic inspection are all presented in Appendix E.

8.1.6 Installation Details

8.1.6.1 WEAP Analysis

Golder carried out Wave Equation Analysis of Pile Driving (WEAP) analysis prior to pile driving to confirm the ability of the proposed pile driving equipment to install the reaction piles to the required tip elevation of -66.0 m geodetic. The analysis was carried out based on the available geotechnical subsurface information and hammer driving (energy) specifications for an APE D180-42 diesel hammer. The results indicated that the proposed APE D180-42 diesel hammer would be suitable for driving the reaction piles to the required tip elevation without damaging/overstressing the reaction piles.

Due to the initial un-availability of FRPD's APE D180-42 diesel hammer at the time of installing the reactions piles, FRPD proposed to use an APE D138-42 diesel hammer to advance the reaction piles as far as practically possible until the originally proposed APE D180-42 was available. Golder carried out additional WEAP analysis for the APE D138-42 hammer using the same parameters that were used for the APE D180-42 diesel hammer. The WEAP analysis indicated that the APE D138-42 would not likely be able to advance the reaction piles fully down to the required -66.0 m tip elevation and that the APE D180-42 would still be required to advance the reaction piles to the required depth as well as conduct the PDA testing.

The results of the WEAP analysis are presented in Appendix F.

8.1.6.2 Installation Dates

The reaction piles were installed concurrently between March 12, 2016 and May 9, 2016. Pile Driving Analyzer testing was carried out on the reaction piles as indicated in Table 4 below:

Table 4: Reaction Pile PDA Test Dates

Activity	RP1	RP2	RP3	RP4
End of Initial Driving	May 9, 2016	May 6, 2016	May 9, 2016	May 6, 2016
One Day Restrike	May 10, 2016	n/a	May 10, 2016	n/a
Three Day Restrike	n/a	May 9, 2016	n/a	May 9, 2016
Seven Day Restrike	May 16, 2016	n/a	May 16, 2016	n/a
Twenty-Nine Day Restrike	June 7, 2016	n/a	June 7, 2016	n/a

8.1.6.3 Pile Driving Equipment Details

Three pieces of pile driving equipment were used to install the reaction piles to the required elevation including an APE 200-6 vibratory hammer, an APE D138-46 diesel hammer and an APE D180-46 diesel hammer. Each of these hammers were used between the approximate tip elevations indicated in Table 5:

Table 5: Pile Driving Hammer Installation Elevations

Pile Driving Hammer	RP1	RP2	RP3	RP4
APE 200-6	+1.1 to -18.1 m	+1.3 m to -7.7 m	+1.1 to -13.0 m	+1.0 to -12.4 m
APE D138-42	-18.1 to -58.0 m	-7.7 to -58.0 m	-13.0 to -57.8 m	-12.4 to -58.0 m
APE D180-42	-58.0 to -66.2 m	-58.0 to -66.2 m	-57.8 to -66.2 m	-58.0 to -66.1 m

The detailed specifications for each of the hammers has been provided in Appendix G. The site-specific operating settings/conditions are provided below:

APE 200-6 Vibrator Hammer:

- Normal Operating Settings; and
- Operating Weight During Pile Driving: 12900 kg (including clamps and hoses).

APE D138-42 Diesel Hammer:

- Operating Weight During Pile Driving: 77600 kg (including leads, strike plate, bell, hoses);
- Fuel Type: Conventional Diesel;
- Fuel Setting 1: (RP3: -13.0 m to -18.7 m; RP4: -12.4 m to -14.4 m);
- Fuel Setting 2: (RP3: -18.7 m to -20.5 m; RP4: -14.4 m to -18.5 m);
- Fuel Setting 3: (RP1: -18.1 m to -19.3 m and -20.2 m to -20.5 m; RP2: -10.4 m to -17.9 m);
- Fuel Setting 4: (RP1: -19.3 m to -20.2 m and -20.5 m to -58.0 m; RP2: -7.7 m to -10.4 m and -17.9 m to -58.0 m; RP3: -20.5 m to -57.8 m; RP4: -18.5 m to -58.0 m); and
- Cushion Type: None (direct drive).

APE D180-42 Diesel Hammer:

- Operating Weight During Pile Driving: 65800 kg (including leads, strike plate, bell, hoses);
- Fuel Type: Conventional Diesel;
- Fuel Setting 4: (RP1: -58.0 m to -66.2 m; RP2: -58.0 m to -66.2 m; RP3: -57.8 m to -66.2 m; RP4 -58.0 m to -66.1 m); and
- Cushion Type: None (direct drive).

FRPD fabricated guide frames at each of the reaction pile locations to precisely locate the reaction piles and maintain their location during pile driving. The guide frames typically consisted of two, 900 mm or 1200 mm diameter open-ended steel piles on either side of the reaction piles with steel I-beam frame assemblies that completely surrounded each pile. The guide piles were installed using the APE 200-6 hammer to approximately 3 m to 4 m depth below the ground surface and the steel I-beam frames were welded onto the guide piles. The guide frame assemblies were completely removed following completion of driving of the reaction piles and the remaining holes were backfilled where required.

8.1.6.4 Splicing Details

The pile splices on the reaction piles were located to permit fresh heading of the pile segments only. On average, approximately 0.3 m was cut off each 11.9 m long pile segment for fresh heading purposes. A total of six pile splices per reaction pile were conducted in the field by FRPD to join all of the segments at the approximate locations indicated in Table 6 below.

Table 6: Reaction Pile Splice Locations

	Pile Segment Length (Splice Distance from Pile Tip)			
Pile Segment	**RP1**	**RP2**	**RP3**	**RP4**
1	11.70 m(11.70 m)	11.70 m (11.70 m)	11.70 m (11.70 m)	11.70 m (11.70 m)
2	11.70 m (23.40 m)	11.50 m (23.20 m)	11.60 m (23.30 m)	11.60 m (23.30 m)
3	11.40 m (34.80 m)	11.40 m (34.60 m)	11.65 m (34.95 m)	11.60 m (34.90 m)
4	11.60 m (46.40 m)	11.60 m (46.20 m)	11.65 m (46.60 m)	2.95 m (37.85 m)
5	3.15 m (49.55 m)	3.34 m (49.54 m)	3.10 m (49.70 m)	11.60 m (49.45 m)
6	11.50 m (61.05 m)	11.64 m (61.18 m)	11.80 m (61.50 m)	11.75 m (61.20 m)
7	9.23 m	9.09 m	8.83 m	9.00 m

The actual as-constructed pile splice locations are indicated on Figure 7.

For each pile splice, the end of the receiving steel segment was cut off perpendicular (90 degrees) to the pile wall and ground clean prior to stacking. The adjoining steel pile section was bevelled and ground clean prior to stacking. A 6 mm thick, 50 mm wide, steel backing plate was provided on the interior wall of pile at each splice location prior to stacking. A multi-pass process was used to complete all splice welding in accordance with CSA W59, by CSA approved welders, and each splice weld was inspected using non-destructive, ultrasonic examination. All test pile field splices met CSA W59 welding requirements.

A copy of the detailed splice weld procedure is provided in Appendix H along with the results of the ultrasonic examinations performed on each splice weld.

8.1.6.5 Pile Driving Records

Detailed field records of activities conducted during installation (driving) of the reaction piles were completed and submitted to MoTI on a daily basis. The daily records typically included the following information:

- Project identification;
- Piling contractor;
- Pile identification;
- Pile type and size;
- Date and time of pile driving;
- Pile length driven, tip elevation, ground elevation;
- Splice locations;
- Elevation of soil inside pile;
- Pile driving start and stop times;
- Rate of penetration (APE 200-6 vibration hammer);
- Blows/0.3 m penetration (APE D138-42 and D180-42 diesel hammers);
- Blows/25 mm penetration at start of driving each pile section (APE D138-42 and D180-42 diesel hammers);
- Blows/minute (APE D138-42 and D180/42 diesel hammers);
- Fuel settings (APE D138-42 and D180/42 diesel hammers);
- Maintenance and standby activities;
- Pile inspection results; and
- Pile Driving Analyzer (PDA) activities.

The daily records for each of the reaction pile installations and PDA testing have been combined to form the official pile driving records for the reaction piles. These daily records are presented in Appendix I.

8.1.6.6 PDA Testing

PDA testing was carried out on each of the reaction piles at the End of Initial Driving (EOID). Two of the reaction piles (RP2 and RP4) had three-day restrikes and the other two (RP1 and RP3) had one-day restrikes, seven-day restrikes, and 29 day restrikes. The dates of all the PDA testing on the reaction pile are presented in Table 4 above. All PDA testing was carried out by MoTI geotechnical services using their own PDA equipment and staff. FRPD pre-drilled holes for the mounting of PDA gauges at locations directed by MoTI. In general, the PDA gauges were located at 5 m below the top of the pile section (minimum 2.5 times the pile diameter) and at least 0.5 m away from existing production welds on the reaction piles.

For the EOID portion of the PDA testing, the PDA recording was carried out continuously between approximate pile tip elevations -58 m and -66 m (geodetic datum). The restrike PDA testing on the reaction piles drove the reaction piles into the ground another 80 mm to 210 mm past the -66 m tip elevation at the EOID, averaging about 155 mm.

MoTI processed the PDA data and provided a summary report of the data collected. The PDA report is presented in Appendix J.

8.1.6.7 Final Tip and Cut-Off Elevation

The final tip and cut-off elevations for the reaction piles were as indicated in Table 7 below. All elevations are referenced to geodetic datum.

Table 7: Reaction Pile Tip and Cut-Off Elevation (Pre Static Load Test)

	RP1	RP2	RP3	RP4
Final Tip Elevation	-66.17 m	-66.16 m	-66.22 m	-66.09 m
Final Cut-Off Elevation	+4.11 m	+4.11 m	+4.11 m	+4.11 m

The as-constructed pile tip and cut-off elevations are presented on Figure 7.

8.2 Test Pile

8.2.1 Pile Type and Size

The test pile was comprised of seven sections of open ended structural steel pipe with nominal dimensions of 2000 mm by 32 mm, welded at designated splice locations. The steel sections were approximately 11.9 m in length (except for one approximate 3.6 m section) and contained both longitudinal and transverse production welds. The steel pipe pile sections all met ASTM A252-10 Grade 3 specifications upon arrival at the site. A total of six 11.9 m long sections were delivered to the site along with a shorter 3.6 m section (76 m approximate length of steel pipe pile prior to pile driving).

The average dimensions of the test pile, based on measurements taken during QC inspection and instrumentation layout were as follows:

- Outside Diameter: 1999 mm
- Thickness: 30.7 mm
- Cross Sectional Area: 190100 mm^2

These dimensional properties have been utilized in assessing the pile loading prior to, during and after the pile load test.

8.2.2 Identification

For reporting purposes, the test pile has been designated as TP1 in this report.

In the quality inspection reporting performed by external parties, and on the Mill Certificates, the test pile has been included in Heat Number 2520208659.

8.2.3 Location

The test pile is located in the center of the test frame/reaction pile assembly as shown in Figure 5. The as-constructed, surveyed (UTM, NAD 83), centroid of the test pile is:

- Northing: 5440753.590
- Easting: 494732.506

8.2.4 Production Dates/Shipping Dates/Mill Certificates

The test pile was manufactured in TianJin, China between October 15, 2015 and November 19, 2015 and was shipped to Port Coquitlam, BC on November 19, 2015.

The Mill Certificate for the test pile is presented in Appendix E (reference Heat Number 2520208659).

8.2.5 Quality Testing/Inspection Results

A Mill Certificate and quality inspection report were provided for the test pile by the pile manufacturer. The Mill Certificate provided by the pile manufacturer indicates that the chemical composition, ultimate tensile strength of the steel and welds, yield strength and dimensional properties all meet ASTM A252, Grade 3 requirements. Further, the quality inspection report prepared by X&C Engineering in China indicated that the dimensional properties, including diameter, ellipticity and straightness, were all acceptable. Visual inspection and random ultrasonic testing was conducted on the productions weld, where defects were noted, they were corrected and re-inspected. There were no non-conformances noted by X&C Engineering for the final test pile product.

Copies of the relevant Mill Certificate and quality inspection report are presented in Appendix E (reference Heat Number 2520208659).

In addition to the above quality documentation, Golder commissioned quality testing on the test pile (and reaction piles) following arrival of the steel pipe in Port Coquitlam, BC. The quality testing included chemical composition testing, yield and ultimate tensile testing on coupons obtained from selected areas of the steel pipe and stress-strain testing of steel coupons obtained from selected portions of production. All of the additional chemical tests demonstrated compliance with CSA Z662-2015 and Z245.1-2014, and all of the additional yield and tensile tests demonstrated compliance with ASTM A252, Grade 3 requirements.

The additional quality inspection reports, including chemical testing, dimensional properties confirmation, coupon testing results and ultrasonic inspection are presented in Appendix E.

8.2.6 Instrumentation Pre-Fabrication

The test pile was required to have a series of vibrating wire strain gauges installed on the perimeter of the pile as well as two telltale rods and housings installed on the perimeter of the pile. Due to the open-ended pile design and necessity to clean the upper portion of the test pile out following installation for placement of concrete, the strain gauges and portions of the telltale instrumentation had to be installed on the outside perimeter of the test pile prior to the test pile being driven below the ground surface.

8.2.6.1 Vibrating Wire Strain Gauges and Protective Plates

The vibrating wire strain gauges (strain gauges) and associated cables were to be installed on four axis, evenly distributed at approximately 90 degrees around the circumference of the test pile. The manufacturer, model and technical specifications of the strain gauges are provided in Section 11.4 below. A total of eight levels of strain gauges, with seven levels located below ground, were required starting at approximately 2 m above the tip of the test pile, then distributed at intervals ranging between about 8.4 m and 13 m along the length of the pile. Each strain gauge level was assigned a number, starting a Level 1 near the bottom of the test pile to Level 8 near the top of the test pile. A minimum of six strain gauges were installed on each below-ground level (Levels 1 through 6), with eight strain gauges installed on Level 7. Only four strain gauges (one on each of the four axes) were installed on Level 8 with no redundant gauges since these gauges were installed after the pile driving was completed. The as-constructed arrangement of the strain gauge installation on the test pile is shown on Figures 6 and 8.

Prior to pile driving below ground, each Levels 1 through 7 were mounted and pre-tensioned on the test pile to within the recommended operating range of the strain gauges (between 1000 and 4000 microstrain) and optimized to remain within this operating range during the Static Load Test (they were pre-tensioned to about 3100 microstrain to allow up to 2100 microstrain in compression, or up to 81MN equivalent load, during pile driving and during the pile load test). The strain gauges were installed with the fixed, or grooved, end of the gauges facing in the upward direction and the adjustable end facing in the downward direction. This was recommended by the manufacturer based on previous case studies of vibrating wire strain gauge failures during pile driving. The adjustable end of the strain gauges was equipped with additional anchoring nuts to provide added protection against loosening during pile driving. The strain gauge cabling was securely tied to tie-down bolts that were pre-welded onto the test pile and, where required, the strain gauges and cabling were wrapped with protective fire-resistant cloth.

Along each of the four axis that the strain gauges were mounted on, a steel protective cover was provided over top of the strain gauges and cabling. The protective cover consisted of a steel 3.5 inch by 3.5 inch by 0.25 inch (88 mm by 88 mm by 6 mm) angle section that was typically pre-welded (continuous welding) onto each of the individual test pile sections prior to hoisting and stacking for splice welding. Selected sections of the protective cover were left off the test pile segments, generally at strain gauge and splice locations, until immediately prior to driving below ground. This allowed for final testing, adjustment, or repair of the strain gauges prior to driving underground (and becoming inaccessible). Where required, the protective cover plate was welded over the gap sections immediately prior to driving underground such that the cover plate extended continuously along each axis from about 1 m above the test pile tip to the ground surface. A 0.6 m long tapered section of cover plate was provided at/near the pile tip to reduce drag resistance during pile driving.

8.2.6.2 Telltale Housing

Two, 25 mm by 25 mm, square steel sections were to be installed onto the test pile starting from about 1.0 m from the test pile tip to about 0.3 m above the ground surface. The square sections were to provide a continuous protective housing for the 9.5 mm diameter stainless steel telltale rods along each side of the test pile. The telltale tube housings were to be installed at diametrically opposed sides of the test pile, at 180 degrees apart, and were to contain a threaded base to secure the stainless steel telltale rods (also threaded) to the base of the test pile.

Similar to the protective steel plate installation for the strain gauges, the telltale housings were pre-installed by continuous welding onto the test pile segments wherever possible prior to hoisting and stacking for splice welding. Gaps were left in the telltale housing installation at splice locations and the gaps were filled following splice welding and inspection. A pre-fabricated, solid steel, tapered tip was installed at the bottom of each telltale housing to cap the end of the telltale housing(s) and reduce the soil drag/resistance at the bottom of the housing(s) during pile driving. These tapered tips were located at 1.0 m distance from the tip of the test pile.

Due to layout and installation problems between the various segments of the test pile, the final arrangement of the telltale housings was such that they were approximately 174 degrees diametrically apart in one direction around the circumference of the test pile and approximately 186 degrees apart in the other direction. Abandoned sections of telltale housing were left remaining on segments 2, 3, 4 and 5 of the test pile. These abandoned sections were approximately 6 degrees clockwise of the telltale TT2 housing (looking down from the top of the test pile) at the following approximate locations:

- 11.9 m and 21.2 m above the pile tip;
- 23.5 m and 32.8 m above the pile tip;
- 35.1 m and 43.1 m above the pile tip; and
- 46.7 m and 54.7 m above the pile tip.

The final as-constructed arrangement of the telltale housings is presented in Figure 6.

8.2.7 Test Pile Installation Details

8.2.7.1 WEAP Analysis

Golder carried out Wave Equation Analysis of Pile Driving (WEAP) analysis prior to pile driving to confirm the ability of the proposed pile driving equipment to install the test pile to the required tip elevation of -66.0 m geodetic. The analysis was carried out based on the available geotechnical subsurface information and hammer driving (energy) specifications for an APE D180-42 diesel hammer. The results indicated that the proposed APE D180-42 diesel hammer would be suitable for driving the test pile to the required tip elevation without damaging/overstressing the test pile.

The results of the WEAP analysis are presented in Appendix F.

8.2.7.2 Installation Dates

The test pile was installed between May 16, 2016 and May 31, 2016. Pile Driving Analyzer (PDA) testing was carried out on the test pile on May 31, 2016 (End of Initial Driving, EOID), June 1, 2016 (One Day Restrike) and June 7, 2016 (Seven Day Restrike).

8.2.7.3 Pile Driving Equipment Details

Two pieces of pile driving equipment were used to install the test pile to the required tip elevation including an APE 200-6 vibratory hammer and an APE D180-46 diesel hammer. The APE 200-6 vibratory hammer was used to install the first segment of the test pile to the desired splice height, which corresponded to a tip elevation of approximately -8.5 m (all elevations are expressed as geodetic datum). The remainder of the test pile was driven to the required tip elevation of -66.0 m using the APE D180-42.

The detailed specifications for each of the hammers has been provided in Appendix G. The site-specific operating settings/conditions for each of the hammers are provided below. The elevations in brackets refer to the approximate tip elevations where each hammer was used:

APE 200-6 Vibrator Hammer (+1.0m to -8.5m):

- Normal Operating Settings; and
- Operating Weight During Pile Driving: 12900 kg (including clamps and hoses).

APE D180-42 Diesel Hammer (-8.5m to -66.1m):

- Operating Weight During Pile Driving: 65800 kg (including leads, strike plate, bell, hoses)
- Fuel Type: Conventional Diesel;
- Fuel Setting 2: (-8.5 m to -20.1 m, -29.8 m to -54.6 m);
- Fuel Setting 3: (-20.1 m to -20.4 m, -56.2 m to -57.1 m);
- Fuel Setting 4: (-20.4 m to -29.8 m, -54.6 m to -56.2 m, -57.1 m to -66.1 m); and
- Cushion Type: None (direct drive).

FRPD fabricated a guide frame at the test pile location to precisely locate the test pile and maintain its location during pile driving. The guide frame consisted of two, 900 mm diameter open-ended steel piles on either side of the test pile and a steel I-beam frame assembly that completely surrounded the test pile. The guide piles were installed using the APE 200-6 hammer to approximately 3 m depth below the ground surface and the steel I-beam frame was welded onto the top of the guide piles. The guide frame assembly was completely removed following completion of driving of the test pile and the remaining holes were backfilled where required.

8.2.7.4 Splicing Details

The pile splice locations were pre-determined and marked on the pile segments to ensure that the strain gauge instrumentation installed on the pile remained at the intended design location(s). Five of the pile sections were cut-off at 11.6 m length, a sixth section was cut-off at 3.365 m length and the seventh (final) section was cut-off at 7.765 m length. A total of six pile splices were conducted in the field by FRPD to join all of the segments. The actual as-constructed pile splice locations are indicated on Figure 6.

For each pile splice, the end of the receiving steel segment was cut off perpendicular (90 degrees) to the pile wall and ground clean prior to stacking. The adjoining steel pile section was bevelled and ground clean prior to stacking. A 6 mm thick, 50 mm wide, steel backing plate was provided on the interior wall of pile at each splice location prior to stacking. A multi-pass process was used to complete all splice welding in accordance with CSA W59, by CSA approved welders, and each splice weld was inspected using non-destructive, ultrasonic examination. All test pile field splices met CSA W59 welding requirements.

A copy of the detailed splice weld procedure is provided in Appendix H along with the results of the ultrasonic examinations performed on each splice weld.

8.2.7.5 Pile Driving Records

Detailed field records of activities conducted during installation (driving) of the test pile were completed and submitted to MoTI on a daily basis. The daily records typically included the following information:

- Project identification;
- Piling contractor;
- Pile identification;
- Pile type and size;
- Date and time of pile driving;
- Pile length driven, tip elevation, ground elevation;
- Splice locations;
- Elevation of soil inside pile;
- Pile driving start and stop times;
- Rate of penetration (APE 200-6 vibration hammer);
- List of instrumentation installed;
- Blows/0.3 m penetration (APE D180-42 diesel hammer);
- Blows/25 mm penetration at start of driving each pile section (APE D180-42 diesel hammer);
- Blows/minute (APE D180/42 diesel hammer);

- Fuel settings (APE D180/42 diesel hammer);
- Maintenance/standby activities;
- Pile inspection results; and
- Pile Driving Analyzer (PDA) activities.

The daily records for each of the seven days of pile driving and two days of PDA testing have been combined to form the official pile driving record for the test pile. These records are presented in Appendix I.

8.2.7.6 PDA Testing

As mentioned above, PDA testing was carried out on the test pile at the End of Initial Driving (EOID), one-day restrike and seven-day restrike. All PDA testing was carried out by MoTI geotechnical services using their own PDA equipment and staff. FRPD pre-drilled holes for mounting of the PDA gauges at locations directed by MoTI. In general, the PDA gauges were located at 5 m below the top of the pile section (minimum 2.5 times the pile diameter) and 0.5 m away from existing production welds on the test pile.

For the EOID portion of the PDA testing, the PDA recording was carried out continuously between pile tip elevations -58.0 m and -66.04 m on May 31, 2016. The one day re-strike PDA testing occurred on June 1, 2016 and the seven day re-strike PDA testing occurred on June 7, 2016. The final pile tip elevation following PDA testing was -66.109 m.

MoTI processed the PDA data and prepared a summary report of the data collected. This PDA report is presented in Appendix J.

8.2.7.7 Final Tip and Cut-Off Elevation

The final tip elevation was -66.106 m geodetic (approximately 67 m below ground surface) and the final pile top cut-off elevation was +3.02 m. Both elevations are reported a geodetic elevations.

The as-constructed pile tip and cut-off elevations are presented on Figure 6.

8.2.8 Test Pile Cleanout and Concrete In-Fill

A detailed procedure for the test pile cleanout and replacement with tremie concrete was developed and submitted to MoTI prior to execution in the field. This procedure has been included in Appendix K.

8.2.8.1 Drill Out

Upon completion of the seven day restrike PDA test and removal of the test pile guide frame, FRPD set up to carry out cleaning of the soil above elevation -20 m geodetic for replacement with tremie concrete. The test pile cleanout activities occurred between June 8 and June 10, 2016 and generally included the following steps:

- Pumping water into the test pile during cleanout to maintain at least 1 m of head above the surrounding ground surface during excavation and cleaning activities;
- Removing the upper portion of the soil plug using a hammer grab style clam bucket mounted on the same crane used for pile installation. This excavation extended down to elevation -20 m, confirmed by hand sounding; and
- Mechanical brushing the inner sidewalls of the test pile using a rotating wire brush mounted on the crane. The mechanical brush was run up and down the inside of the pile down to the bottom of the excavation depth at least five times until the brushes and sidewalls were clean. The sidewalls were reasonably clean prior to brushing as there were no cohesive soils within the soil plug and the mechanical washing created during excavation effectively cleaned the non-cohesive soils from the sidewalls.

8.2.8.2 Concrete In-Fill Specification

Following completion of the cleanout activities, the cleaned out portion of the test pile was filled with tremie concrete. The tremie concrete specification called for a non-shrink type concrete with a minimum 14 day compressive strength of at least 30 MPa. The concrete mix design was submitted to the Ministry prior to concrete placement and is included in Appendix K.

8.2.8.3 Concrete In-Fill Test Results

Concrete placement occurred on June 14, 2016 and concrete sampling and field testing was carried out throughout the day during concrete placement. A total of 8 test cylinders were cast to allow compressive strength testing at 14 days, 28 days and on the day of the pile load test. The compressive strength testing on the test cylinders included stress-strain measurements to allow estimation of the elastic modulus of the concrete mass. All test cylinders were cured in plastic cylinders in one of the site office trailers for 24 hours and then taken to Metro Testing Laboratories in Burnaby and cured in a moisture and temperature controlled environment.

A summary of the concrete testing results is presented in Table 8 below:

Table 8: Summary of Concrete Testing Results

Cylinder Casting Date	Test Break Date	Elapsed Time	Compressive Strength (MPa)	Elastic Modulus (GPa)
June 14, 2016	June 28, 2016	14 day	50.8	28.7
June 14, 2016	June 28, 2016	14 day	46.4	28.9
June 14, 2016	July 12, 2016	28 day	52.5	33.3
June 14, 2016	July 12, 2016	28 day	55.9	32.1
June 14, 2016	August 18, 2016	65 day	53.7	30.9
June 14, 2016	August 18, 2016	65 day	63.2	39.0

The concrete testing reports for each of the test cylinders are presented in Appendix K.

8.2.8.4 Concrete In-Fill Elevations

The final concrete in-fill elevations prior to the Static Load Test were as follows:

- Bottom of concrete in-fill: -20.0 m geodetic; and
- Top of concrete in-fill: 0.0 m geodetic.

It is noted that following the initial and second loading and unloading sequences on August 18 and 19, 2016, MoTI gave direction to infill the upper portion of the test pile from the top of the previous concrete infill to the top of the test pile (underside of the load plate). This additional concrete infill was placed on August 25, 2016 using the same concrete mix design as the previous concrete infill as directed by MoTI. No concrete sampling and/or testing was conducted during placement of the concrete mix.

8.3 Pile Driving Impacts

8.3.1 Pile Driving Monitoring Results

8.3.1.1 Survey Monuments

A comprehensive set of survey data was obtained for all of the survey monuments including collection of the required baseline data (1 day), the duration of pile driving and the required monitoring period (5 days) following pile driving.

Overall, the measured absolute displacements between the initial baseline readings and post pile driving were relatively small, typically within the minimum survey accuracy and did not exceed 10 mm in any direction for any survey monument. However, much more significant displacements were measured during pile driving activities due to the dynamic shock wave propagation generated during pile driving. Some of the highest measured displacements approached 25 mm to 50 mm; however, these measurements were discontinuous (presented as spikes on time lapse graph) and quickly rebounded to at, or near, the average readings measured before daily pile driving activities.

A summary of the measured absolute displacements for each of the survey monuments is provided in Table 9 below.

Table 9: Absolute Displacements of Survey Monuments

Point ID	Time	Target Easting [m]	Target Northing [m]	Target Elevation [m]	Longitudinal Displacement [m]	Transverse Displacement [m]	Height Displacement [m]
33	07/06/2016 12:49	494700.46	5440745.6	2.33	-0.004	0.009	0.001
J1-1	07/06/2016 12:50	494695.01	5440756.8	-11.581	0	0.002	-0.007
J1-2	07/06/2016 12:50	494692.17	5440760.4	-12.749	0.001	0.001	-0.006
J2-1	07/06/2016 12:49	494629.65	5440837.9	-14.462	0	0	0.002
J2-2	07/06/2016 12:51	494625.34	5440843.3	-17.405	0	-0.001	0.002
SME01	07/06/2016 12:49	494700.38	5440754.4	4.239	0.003	0.006	0.004
SME02	07/06/2016 12:50	494704.47	5440749.3	4.263	-0.002	0.007	0.001
SME03	07/06/2016 12:49	494701.44	5440745.3	1.513	-0.001	0.009	0
SME04	07/06/2016 12:50	494708.38	5440736.7	1.506	0.002	0.008	-0.001
SME05	07/06/2016 12:50	494709.17	5440735.8	1.497	0.005	0.009	-0.001
SME06	07/06/2016 12:49	494717.29	5440725.7	1.526	0.002	0.005	0.003
SME07	07/06/2016 12:50	494718.04	5440724.7	1.514	0.007	0.007	0
SME08	07/06/2016 12:50	494726.19	5440714.6	1.501	0.002	0.003	0.003
SME09	07/06/2016 12:50	494726.82	5440713.8	1.515	0.001	0.001	0.001
SME10	07/06/2016 12:50	494735.04	5440703.6	1.521	0.001	0.001	0.004
SME11	07/06/2016 12:50	494735.73	5440702.7	1.518	0.001	0	0.001
SMW01	07/06/2016 12:49	494678.94	5440737.2	4.235	0.002	0.005	0.005
SMW02	07/06/2016 12:50	494683.02	5440732	4.255	0.003	0.004	0.005
SMW03	07/06/2016 12:50	494687.64	5440734.1	1.499	0	0	0.005
SMW04	07/06/2016 12:50	494694.49	5440725.6	1.546	0.002	0.001	0.006
SMW05	07/06/2016 12:48	494695.29	5440724.7	1.532	0	0	0.005
SMW06	07/06/2016 12:49	494703.48	5440714.6	1.525	0.002	0	0.007
SMW07	07/06/2016 12:50	494704.23	5440713.7	1.519	0.001	-0.001	0.008
SMW08	07/06/2016 12:49	494712.41	5440703.5	1.535	0.002	0.002	0.007
SMW09	07/06/2016 12:50	494713.12	5440702.6	1.546	0	-0.001	0.009
SMW10	07/06/2016 12:49	494721.23	5440692.6	1.534	0.001	0.004	0.008
SMW11	07/06/2016 12:51	494722.01	5440691.6	1.539	0	-0.001	0.009

More detailed plots displaying the measured daily cumulative displacement for each of the survey monuments between acquisition of the initial baseline readings and the acquisition of the post pile driving measurements are presented in Appendix L. The detailed survey results have been provided to MoTI in the previous daily inspection report submission.

8.3.1.2 Joint Meters

A comprehensive set of joint meter data was obtained including collection of the required baseline data (1 day), the duration of pile driving and the required monitoring period (5 days) following pile driving.

Overall, the measured joint displacements between the initial baseline readings and post pile driving were relatively small, typically between about 2 mm and 5 mm. However, upon examination of cumulative displacement plots over the duration of pile driving, the measured displacements appear to coincide with seasonal temperature fluctuations at the site. At no time, during pile driving activities, did it appear that there was any significant relative movement between any of the approach wall sections. This was confirmed by visual observation at the site.

A summary of the measured cumulative displacements for each of the joint meters is provided in Table 10 below.

Table 10: Absolute Displacements of Joint Meters

Time	JM1 (mm)	JM2 (mm)	JM3 (mm)	JM4 (mm)	JM5 (mm)	Comment
09/03/2016 0:00	-0.3	0.0	0.1	0.1	0.0	Prior to start of pile-driving
07/06/2016 16:54	-4.1	-4.8	-1.9	-1.7	-2.0	After completion of pile-driving

More detailed plots displaying the measured daily cumulative displacement for each of the joint meters between acquisition of the initial baseline readings and the acquisition of the post pile driving measurements are presented in Appendix L.

8.3.1.3 Deep Settlement Gauges

A comprehensive set of deep settlement gauge data was obtained including collection of the required baseline data (1 day), the duration of pile driving and the required monitoring period (5 days) following pile driving. It is noted that only vertical displacement of the deep settlement gauges was measured using a conventional laser level system following pile driving; however, no discernable vertical displacement of the deep settlement gauges was measured following pile driving.

The cumulative vertical displacement of the deep settlement gauges typically ranged between about 8 mm and 25 mm with the greatest amount of displacement occurring at DSG2 (closest to reaction pile RP4).

A summary of the measured vertical cumulative displacements for each of the deep settlement gauges is provided in Table 11 below.

Table 11: Absolute Vertical Displacements of Deep Settlement Gauges

Date	DSG1 Cumulative Settlement (m)	DSG2 Cumulative Settlement (m)	DSG3 Cumulative Settlement (m)	DSG4 Cumulative Settlement (m)	DSG5 Cumulative Settlement (m)
09-Mar-16	0	0	0	0	0
12-Mar-16	0	-0.001	-0.001	0	0
16-Mar-16	0	-0.001	-0.001	-0.001	-0.001
30-Mar-16	0	-0.001	-0.001	-0.002	-0.001
31-Mar-16	-0.001	-0.001	-0.001	-0.002	-0.001
01-Apr-16	-0.001	-0.001	-0.001	-0.003	-0.001
05-Apr-16	-0.001	-0.001	-0.001	-0.003	-0.001
11-Apr-16	-0.001	-0.001	-0.001	-0.003	-0.001
12-Apr-16	-0.004	-0.013	-0.003	-0.008	-0.005
13-Apr-16	-0.004	-0.013	-0.003	-0.008	-0.006
14-Apr-16	-0.005	-0.014	-0.005	-0.01	-0.007
15-Apr-16	-0.005	-0.015	-0.005	-0.01	-0.008
19-Apr-16	-0.005	-0.017	-0.007	-0.011	-0.006
21-Apr-16	-0.006	-0.019	-0.009	-0.012	-0.008
05-May-16	-0.006	-0.019	-0.008	-0.012	-0.007
06-May-16	-0.005	-0.019	-0.007	-0.013	-0.008
09-May-16	-0.006	-0.02	-0.008	-0.012	-0.008
10-May-16	-0.006	-0.021	-0.009	-0.015	-0.009
11-May-16	-0.006	-0.021	-0.009	-0.015	-0.009
12-May-16	-0.007	-0.021	-0.009	-0.015	-0.009
16-May-16	-0.007	-0.021	-0.010	-0.015	-0.009
18-May-16	-0.007	-0.021	-0.011	-0.015	-0.009
20-May-16	-0.007	-0.022	-0.011	-0.015	-0.009
25-May-16	-0.007	-0.022	-0.012	-0.015	-0.009
27-May-16	-0.007	-0.023	-0.012	-0.015	-0.009
31-May-16	-0.008	-0.023	-0.013	-0.015	-0.009
01-Jun-16	-0.008	-0.023	-0.013	-0.015	-0.009
07-Jun-16	-0.008	-0.025	-0.013	-0.016	-0.009

Detailed plots displaying the measured daily cumulative vertical displacement for each of the deep settlement gauges between acquisition of the initial baseline readings and the acquisition of the post pile driving measurements are presented in Appendix L.

8.3.1.4 Vibration Sensors

A comprehensive set of vibration monitoring data was obtained including collection of the required baseline data (1 day), the duration of pile driving and the required monitoring period (5 days) following pile driving.

The maximum peak particle velocity (PPV) values recorded at each of the geophone locations generally ranged between about 0.5 mm/s and 18 mm/s with the greatest amount of displacement occurring at VS5 (closest to reaction pile RP3).

A summary of the measured vertical cumulative displacements for each of the deep settlement gauges is provided in Table 12 below.

Table 12: Maximum Peak Particle Velocities

	Maximum Peak Particle Velocity Recorded (mm/s)						
Date	**VS1**	**VS2**	**VS3**	**VS4**	**VS5**	**VS6**	**Site Activity**
09-Mar	1.833	1.298	2.21	1.634	2.883	2.551	Guide Pile Installation
12-Mar	2.16	0.506	1.76	4.48	3.55	3.18	APE 200-6 Vibratory Driving
16-Mar	8.94	0.876	10.4	7.16	14.9	13.4	APE 200-6 Vibratory Driving
30-Mar	16.4	6.82	8.71	11.3	15.1	26.7	APE D138-42 diesel hammer
31-Mar	5.15	7.53	9.76	13.2	15.2	6.89	APE D138-42 diesel hammer
01-Apr	5.59	8.46	3.97	14.2	9.41	8.31	APE D138-42 diesel hammer
05-Apr	4.58	6.84	6.74	9.95	7.89	8.35	APE D138-42 diesel hammer
11-Apr	3.636	5.867	6.921	8.172	7.369	4.324	APE D138-42 diesel hammer
12-Apr	12.99	6.608	8.522	12.74	17.98	19.43	APE D138-42 diesel hammer
13-Apr	4.545	1.981	4.047	4.132	6.278	7.323	APE D138-42 diesel hammer
14-Apr	9.82	6.399	6.65	9.005	9.32	15.82	APE D138-42 diesel hammer
15-Apr	8.652	5.829	7.126	10.56	7.889	12.46	APE D138-42 diesel hammer
19-Apr	8.954	6.523	6.104	9.765	10.6	13.3	APE D138-42 diesel hammer
21-Apr	4.97	4.71	5.18	8.52	7.51	8.12	APE D180-42 diesel hammer
05-May	3.32	3.66	2.74	3.4	4.93	6.18	APE D180-42 diesel hammer
06-May	2.84	10.5	5.37	12.74	13.94	4.8	APE D180-42 diesel hammer
09-May	6.57	10.38	5.28	12.82	13.81	11.54	APE D180-42 diesel hammer
10-May	6.19	2.83	3.42	4.54	6.45	10.53	APE D180-42 diesel hammer
11-May	1.28	0.48	1.00	2.42	2.56	2.19	Guide Pile Installation
12-May	1.81	0.7	1.81	2.81	5.72	3.34	Guide Pile Installation
16-May	5.07	9.9	3.15	7.51	11	9.49	APE D180-42 diesel hammer
18-May	5.38	7.39	4.05	6.25	9.24	6.28	APE D180-42 diesel hammer
20-May	6.35	8.2	4.24	7.52	7.22	5.16	APE D180-42 diesel hammer
25-May	3.9	5.45	5.01	5.45	5.96	4.92	APE D180-42 diesel hammer
27-May	3.11	4.7	4.45	4.48	5.56	3.48	APE D180-42 diesel hammer
31-May	2.81	3.74	3.99	4.05	6.07	4	APE D180-42 diesel hammer
01-Jun	2.87	3.45	4.09	4.69	4.48	2.97	APE D180-42 diesel hammer
07-Jun	5.45	3.8	3.17	5.5	9.55	11.8	APE D180-42 diesel hammer

More detailed plots displaying the measured daily maximum PPV value for each of the vibration geophones between acquisition of the initial baseline readings and the acquisition of the post pile driving measurements are presented in Appendix L. In addition, detailed plots displaying the maximum measured vibrations for each of the installed piles by depth and pile segment are presented in Appendix L.

8.3.1.5 Noise Meters

A comprehensive set of noise monitoring data was obtained including collection of the required baseline data (1 day), the duration of pile driving and the required monitoring period (5 days) following pile driving.

The maximum Lmax values recorded at each of the noise monitoring locations generally ranged between about 99 and 121 dBA during pile driving activities. Similarly, the maximum Leq and Lpeak values recorded generally ranged between about 73 dBA and 97 dBA, and between about 122 dBA and 134 dBA respectively.

A summary of the maximum Lmax, Leq and Lpeak levels measured during pile diving is provided in Table 13 below.

Table 13: Maximum Noise Measurements During Pile Driving

Date	Max Noise Meter Recording (Lmax -dBA)	Noise Meter Location (Max)	Max Noise Meter Recording (Leq -dBA)	Noise Meter Location (Max)	Max Noise Meter Recording (Lpeak -dBA)	Noise Meter Location (Max)	Site Activity
09-Mar	109.4	Site 16b	73.7	Site 17	121.9	Site 15	Guide Pile Installation
12-Mar	99.4	Site 17	75.7	Site 17	121.5	1PLT	APE 200-6 Vibratory Driving
16-Mar	107.5	Site 17	79.4	Site 15c	123.3	1PLT	APE 200-6 Vibratory Driving
30-Mar	113	Site 17	76.1	1PLT	122.8	Site 15	APE D138-42 diesel hammer
31-Mar	110.2	Site 17	83.8	1PLT	126.4	1PLT	APE D138-42 diesel hammer
01-Apr	117.5	1PLT	86.3	1PLT	131.1	1PLT	APE D138-42 diesel hammer
05-Apr	120.9	1PLT	91.8	1PLT	131.8	1PLT	APE D138-42 diesel hammer
11-Apr	119.7	1PLT	94.3	1PLT	131.8	1PLT	APE D138-42 diesel hammer
12-Apr	115.8	1PLT	96.9	1PLT	128.4	1PLT	APE D138-42 diesel hammer
13-Apr	111.5	1PLT	80.2	1PLT	126.6	1PLT	APE D138-42 diesel hammer
14-Apr	112.6	1PLT	93.7	1PLT	127.3	1PLT	APE D138-42 diesel hammer
15-Apr	116.1	1PLT	92.2	1PLT	130.2	1PLT	APE D138-42 diesel hammer
19-Apr	117.4	1PLT	96.2	1PLT	130.2	1PLT	APE D138-42 diesel hammer
21-Apr	114.1	1PLT	95.9	1PLT	127.8	1PLT	APE D180-42 diesel hammer
05-May	114.1	1PLT	83.3	1PLT	130.9	1PLT	APE D180-42 diesel hammer
06-May	120.5	1PLT	96.4	1PLT	133.5	1PLT	APE D180-42 diesel hammer
09-May	113	1PLT	91.6	1PLT	129	1PLT	APE D180-42 diesel hammer
10-May	108.2	1PLT	74	1PLT	124.4	1PLT	APE D180-42 diesel hammer
16-May	114.3	1PLT	86.9	1PLT	128.5	1PLT	APE D180-42 diesel hammer
18-May	111.9	1PLT	90.5	1PLT	133	1PLT	APE D180-42 diesel hammer
20-May	115.7	1PLT	93.5	1PLT	131	1PLT	APE D180-42 diesel hammer
25-May	114.8	1PLT	89.2	1PLT	132.2	1PLT	APE D180-42 diesel hammer
27-May	108.4	1PLT	83.7	1PLT	124.7	1PLT	APE D180-42 diesel hammer
31-May	115.3	1PLT	93.5	1PLT	131.9	1PLT	APE D180-42 diesel hammer
01-Jun	110.5	1PLT	72.7	1PLT	126.4	1PLT	APE D180-42 diesel hammer
07-Jun	108.5	1PLT	71.5	1PLT	125.8	1PLT	APE D180-42 diesel hammer

More detailed plots displaying the measured noise levels at each of the noise meter locations by day during pile driving are presented in Appendix L.

8.3.1.6 Ground Surface Monitoring

A comprehensive set of ground surface monitoring data was obtained for the ground survey points including a collection of the required baseline data (1 day) and the duration of pile driving.

The measured vertical displacements between the initial baseline readings and post pile driving varied significantly across the ground surface monitoring area, ranging between about 0.03 m and 0.47 m.

A summary of the vertical displacements for each of the ground surface monitoring locations between the initial baseline reading on January 11, 2016 and the final day of Pile Driving on June 7, 2016 is provided in Table 14 below.

Table 14: Measured Vertical Displacement at Ground Surface Monitoring Locations

Survey Point Reference	Ground Elevation (m)	Net Elevation Change Since Baseline Reading (m)
GP2+36	1.58	-0.07
GP2+33	1.22	-0.22
GP2+30	CNS	CNS
CP2+09	0.62	-0.47
GP2+06	0.98	-0.13
GP2+03	1.01	-0.18
GP2E02	1.02	-0.07
GP2W02	0.76	-0.06
GPT+36	CNS	CNS
GPT+33	1.10	-0.16
GPT+30	CNS	CNS
GPT+09	0.77	-0.13
GPT+06	0.95	-0.11
GPT+03	0.94	-0.15
GP2E04	1.46	-0.04
GP2W04	0.92	-0.05
GP1+36	CNS	CNS
GP1+33	0.80	-0.46
GP1+30	CNS	CNS
GP1+09	0.49	-0.44
GP1+06	0.63	-0.24
GP1+03	1.12	0.03
GP2E06	1.88	-0.03
GP2W06	1.00	-0.03
ManH	1.37	0.00

Notes: CNS denotes 'Could Not Survey'.

The baseline reading for reference point ManH was taken on March 29, 2016 and the baseline reading for reference points GP2E and GP2W were taken on April 12, 2016.

8.3.2 Post-Construction Inspection

All-Span carried out a visual post-construction condition assessment of the adjacent structures that were assessed during the pre-construction condition visual assessment (refer to Section 7.2 above). Based on the results of the post-condition assessment, it was determined that the condition of the tunnel structural elements were very similar, almost unchanged, to the condition observed during the pre-construction assessment. The results of All-Span's post-construction condition assessment are presented in Appendix C.

Although no discernable structural damage was recorded in the post-construction inspection, significant permanent disturbance to the areas immediately surrounding the pile installation(s) was noted during pile driving with the APE 200-6 vibratory hammer. The area located between the eastern approach wall and crane pad experience significant liquefaction of the sub-surface soils during vibratory pile driving which created subsidence of the ground surface and flotation of some of the existing underground facilities. Of particular note, the existing concrete junction chamber located within this area appeared to have heaved upward, likely due to flotation during liquefaction. Upon inspection of the junction chamber and lower areas in the Massey Tunnel where drainage piping from the junction chamber discharges, it was observed that water and sand was flowing into the chamber and through the drainage piping down into the maintenance tunnel. The sand was cleaned out from the bottom of the tunnel by Mainroad.

Ground liquefaction in this area appeared to stop when the vibratory pile driving ceased and the diesel hammer pile driving started. There was no further water and/or sand flow observed into the junction chamber or Massey Tunnel and further ground surface disturbance was not observed.

9.0 LOAD FRAME

9.1 Background

The load frame used for the static load test was the same load frame that was used for pile load testing on both the recent Pitt River Bridge and Port Mann Bridge Replacement Projects. Due to the different test pile, reaction pile and load frame sizes/configurations used for those projects, the load frame, in its previous configuration, did not meet the minimum spacing dimensions required for the Massey Tunnel Replacement Project Pile Load Test. As such, the load frame required some structural modification to meet the minimum spacing dimensions consistent with ASTM D1143M-07 specifications (a minimum of 5 pile diameters needed to be maintained between the test pile and each of the reaction piles for this Static Load Test).

The previous loading history of the load frame was confirmed prior to structural re-design. Based on the previous pile load test results obtained at both the Pitt River Bridge and Port Mann Bridge sites, it is understood that the load frame had previously been loaded to at least 44.9 MN and 53.7 MN reaction forces, respectively.

9.2 Design

The original load frame was designed by All-Span, and All-Span was again retained to complete the re-design of the load frame to meet the requirements of the Massey Tunnel Replacement Project Pile Load Test. In general, the re-design consisted of lengthening the center load beam to accommodate the minimum spacing requirements of the Static Load Test while retaining the reaction beams at their original dimensions. The center load beam was extended by adding new beam segments on each end of the original load beam and connecting the three segments with slip-critical, bolted connections on the load beam webs and flanges. The ends of the extended load beam were connected to the reaction beams by bolted connections and the reaction beams were connected to the reaction piles by welded connections. Other structural modifications included thickening of some of the flanges by welding on doubler plates.

The modified load frame was designed to resist up to 50 MN vertical load during the pile test (the minimum load capacity required in the RFP/contract). The detailed As-Constructed drawings of the Load Frame Assembly are presented in Appendix M.

9.3 Structural Modifications

As mentioned above, the main structural modifications to the load frame assembly included extension of the center load beam on each end and a new connection design onto the reaction piles. The load beam extension(s) required new spice plates, doubler plates and high tensile strength bolts to physically connect the load beam sections and stiffen the center load beam. The reaction pile connection required vertical steel plates welded onto the top of the reaction piles and onto the web of the reaction beam(s).

There were no other structural modifications required to construct the load frame structure.

9.4 Load Frame Assembly

Portions of the newly modified test frame were pre-assembled in MacForm's construction yard in Langley, BC to test fit some of the frame components; however, the entire load frame could not be completely pre-assembled in the yard due to lifting and space constraints.

The load frame pieces were transported to the Massey Tunnel Replacement Project Test Pile site on July 5, 2016 and assembly started on July 6, 2016. The load frame was assembled using a mobile, 200 tonne crane in the following general sequence:

- The center piece of the load beam (Section A) was placed on the test pile jacking assembly;
- The north piece of the load beam (Section B - North) was placed on a temporary Lock Block pedestal with hydraulic jacking system and the web splice plates were bolted on to partially join Sections A and B - North;
- The south piece of the load beam (Section B - South) was placed on a temporary Lock Block pedestal with hydraulic jacking system and the web plates were bolted on to partially join Sections A and B - South;
- The north reaction beam was placed on the reaction piles and bolted to the north end of the load beam;
- The south reaction beam was placed on the reaction piles and bolted to the south end of the load beam;
- The load frame sections were aligned to within the structural design tolerances and fit to field conditions;
- The reaction beam to reaction pile connector plates were mounted and welded into place;
- The flange splice plates on the load beam were mounted and bolted together; and
- The entire load frame was pre-tensioned using the test pile hydraulic jacking system to remove as much slack from the frame assembly as possible, then all the bolts were torqued to structural specifications.

The load frame assembly was monitored by Golder to verify compliance with structural design requirements and site safety protocols and verify that the necessary periodic field reviews were being carried out by the structural engineering team (All-Span). Apart from two non-conformances that were corrected in the field, the load frame was constructed consistent with the design requirements. All-Span provided a letter confirming that the load frame was constructed in conformance with the design and that the load frame was "fit for purpose" on August 17, 2016. This letter is provided in Appendix M. The load frame assembly was fully completed on August 17, 2016.

10.0 JACKING SYSTEM

10.1 Design

The jacking system was designed by All-Span as an integral part of the pile load application process which included consideration of the comprehensive load frame assembly, jacking assembly, load plate and spacer plate assemblies, load cell assembly and test pile. The jacking system was designed to apply the test loads at the required loading intervals while not overstressing any of the structural elements (including the test pile). The jacking system was also designed to optimize the useable stroke capacity of the jacks (about 123 mm each) and shimming plates (about another 300 mm thickness) for an overall stroke capacity of about 423 mm. Various combinations of loading cycles were considered in the jack shimming, pumping and sequencing design to account for various loading/deflection scenarios that could potentially occur during the Static Load Test. A jacking flow diagram, presenting the various combinations of anticipated loading cycles, is presented in the test frame As-Constructed drawings presented in Appendix M.

A total of twelve hydraulic jacks were used in the jacking system that were aligned in a symmetrical, circular layout between the test pile and load frame and twelve load cells (discussed in Section 10.1 below) were placed immediately below the hydraulic jacks and a steel spacer place. To maintain stability of the jacking and load cell arrangement and to adequately reduce bearing loading on the load frame and test pile, three - 2.34 m diameter solid steel load plates were provided including areas between the load cells and test pile (152 mm thick plate), the load cells and the hydraulic jacks (38 mm thick plate) and the hydraulic jacks and load frame (152 mm thick plate). The hydraulic jacks and load cells were arranged in a circular pattern at/near the outside of the steel plates at 30 degree intervals.

The hydraulic jacks were connected to a variable pumping and valve system which allowed simultaneous jacking and draining of up to six jacks at time (this allowed for quick shimming process below 30 MN load) and allowed for jacking of up to 10 jacks at a time (for loading above 30 MN). Shimming at the higher loads, where required, required test loads to be held constantly while draining/shimming two pumps at a time (a process that needed to be repeated five times each time shimming was required above 30 MN). A detailed procedure for the jacking sequencing was produced by All-Span to account for the various loading scenarios that could potentially occur during the Static Load Test. This procedure is presented in Appendix N.

10.2 Jack and Pump Specifications

The hydraulic jacks used in the Static Load Test were SPX Power Team Model R5656L single acting, load return locking collar hydraulic cylinders. Each of the hydraulic jacks had a lifting capacity of about 5 MN (565 tons) with a 150 mm (6”) stroke; however, based on the advice of the jack operator and structural engineer, the actual useable stroke of the hydraulic jacks was actually about 123 mm (5”) and this was accounted for in the jacking design. The hydraulic pressure range for the jacks ranged between 0 and 69 MPa (0 to 10,000 psi).

Two Simplex G5 Series electric hydraulic pumps connected to two manifolds and multiple shutoff valves provided the necessary hydraulic pressure to the system.

The detailed manufacturer specifications for the hydraulic jacks and pumps used in the Static Load Test have been included in Appendix N.

10.3 Calibration Reports

Each of the hydraulic jacks was calibrated prior to arrival on site. A copy of the jack calibration records is provided in Appendix N.

10.4 Operation

Hymach Industries was retained to set up the jacking and pumping system and operate the jacks during the Static Load Test. Below 30 MN test load, only six of the twelve jacks applied the test load at any given time, while the remaining six where being drained and/or shimmed. There were two pumps in the system so one pump could apply pressure to six of the jacks while one could drain the remaining six jacks where needed. Two individual manifolds (one on each pump) ensured that the pump pressures would be equal. The unloaded jacks were continuously shimmed (by hand) while the other jacks were in operation to reduce the shimming time needed when the loaded jacks ran out of stroke. Above 30 MN, the two manifolds were connected to a single pump to ensure even pressure in the system and individual valves to each jack were used to apply/hold load and/or drain the jacks for shimming.

The hydraulic pumps and shut off valves were manually operated during the test. The operators used the pressure gauges to each pump and the load cell readings to apply and maintain the hydraulic pressure in the system (and load on the test pile). The corresponding load vs. hydraulic pressure readings were calculated prior to conducting the Static Load Test. Table 15 below presents the gauge pressures reading vs. applied load based on the jack calibration records.

Table 15: Gauge Pressure Reading for Applied Load (psi)

Applied Load on Pile (MN)	Number of Jacks Engaged		
	6	10	12
2	663	398	331
4	1,325	795	663
6	1,988	1,193	994
8	2,650	1,590	1,325
10	3,313	1,988	1,656
12	3,975	2,385	1,988
14	4,638	2,783	2,319
16	5,301	3,180	2,650
18	5,963	3,578	2,982
20	6,626	3,975	3,313
22	7,288	4,373	3,644
24	7,951	4,770	3,975
26	8,613	5,168	4,307
28	9,276	5,566	4,638
30	9,939	5,963	4,969
32	n/a	6,361	5,301
34	n/a	6,758	5,632
36	n/a	7,156	5,963
38	n/a	7,553	6,294
40	n/a	7,951	6,626
42	n/a	8,348	6,957
44	n/a	8,746	7,288
45	n/a	8,945	7,454

10.5 Loading Sequence

The actual loading sequence and time duration at each load increment used in the Static Load Test is indicated in Table 16 below.

Table 16: Static Load Test Loading and Unloading Sequence

Load Increment	Loading Curve	Applied Load (MN)	Planned Hold Duration
1	First Load	2	15 min
2	First Load	4	15 min
3	First Load	6	15 min
4	First Load	8	15 min
5	First Load	10	15 min
6	First Load	12	15 min
7	First Load	14	15 min
8	First Load	16	15 min
9	First Load	18	15 min
10	First Load	20	15 min
11	First Load	21	15 min
11	First Load	22	15 min
11	First Load	24	15 min
12	First Load	26.2	15 min
13	First Unload	23	10 min
14	First Unload	20	5 min
15	First Unload	17	5 min
16	First Unload	14	5 min
17	First Unload	11	5 min
18	First Unload	8	5 min
19	First Unload	5	5 min
20	First Unload	2	5 min
22	First Unload	0	30 min
23	Second Load	4	10 min
24	Second Load	8	10 min
25	Second Load	12	10 min
26	Second Load	16	10 min
27	Second Load	20	10 min
28	Second Load	24	10 min
29	Second Load	26.7	10 min
30	Second Unload	21	5 min
31	Second Unload	16	5 min
32	Second Unload	11	5 min
33	Second Unload	6	5 min

Load Increment	Loading Curve	Applied Load (MN)	Planned Hold Duration
34	Second Unload	0	30 min
35	Third Load	2	15 min
36	Third Load	4	15 min
37	Third Load	6	15 min
38	Third Load	8	15 min
39	Third Load	10	15 min
40	Third Load	12	15 min
41	Third Load	14	15 min
42	Third Load	16	15 min
43	Third Load	18	15 min
44	Third Load	20	15 min
45	Third Load	21	15 min
46	Third Load	22	15 min
47	Third Load	24	15 min
48	Third Load	26	15 min
49	Third Load	28	15 min
50	Third Load	29.1	1 min
51	Third Unload	25	5 min
52	Third Unload	22	5 min
53	Third Unload	19	5 min
54	Third Unload	16	5 min
55	Third Unload	13	5 min
56	Third Unload	10	5 min
57	Third Unload	7	5 min
58	Third Unload	4	5 min
59	Third Unload	1	5 min
60	Third Unload	0	30 min

11.0 PILE TEST INSTRUMENTATION

The purpose of the pile test instrumentation and data collection system is to obtain a set of pile load and deformation data in accordance with ASTM D1143M-07 – "Standard Test Methods for Deep Foundations Under Static Axial Compressive Load". As described above, the data will be obtained during hydraulic jack load application onto the instrumented test pile against an anchored test frame. The follow sections provide a summary of the pile test instrumentation used for the Static Load Test.

11.1 Load Cells

Twelve load cells were mounted directly on top of the test pile load plate, approximately in line with each of the hydraulic jacks. The load cells were mounted evenly around the circumference of the test pile, at 30 degree intervals, approximately in-line with the wall of the test pile. The load cells were supplied by RST Instruments, with the following properties:

- Operating range: 0 to 5MN each, or 0 to 60MN for the combined twelve load cells; and
- Precision: ± 0.02% (1 kN).

Each of the load cells were calibrated prior to delivery to the site. The calibration records are provided in Appendix O.

The twelve load cells were connected to the Automated Data Acquisition System (ADAS) which was configured to read the twelve load cells and display the combined load reading at two second intervals. This provided real-time display of applied load to the jack operators during the Static Load Test.

11.2 Liquid Level Gauges

A liquid level system consisting of two transducers capable of measuring water pressure (head) in a common liquid reservoir was installed to precisely measure pile head deflection during the Static Load Test. One of the liquid level gauge transducers was mounted directly on the north side test pile at/near the existing ground surface with another reference liquid level gauge transducer mounted on the reference H-pile (discussed in Section 10.6 below). A small, 0.5 m deep localized excavation was provided immediately below the transducer mounted on the test pile to accommodate pile/liquid level gauge movement during the Static Load Test. The liquid level reservoir connecting the transducers consisted of a 75 mm diameter flexible polyethylene pipe that covered with mineral soil between the test pile and the reference H-pile to reduce potential temperature change effects. The liquid used in the system was 100% water and completely filled the reservoir piping system.

The liquid level system was custom designed and supplied by RST Instruments and had the following properties:

- Operating range: 0 to 500 mm;
- Precision: ± 0.1 mm; and
- Temperature operating range: 0ºC to +40ºC.

Each of the liquid level gauge transducers were calibrated prior to delivery to the site. The calibration records are provided in Appendix O.

The liquid level gauge transducers were connected to the Automated Data Acquisition System (ADAS) which was configured to read the transducers at two second intervals.

11.3 Laser Tracking Survey

A precision laser tracking system was used to monitor and record absolute movement of the above ground portions of the test pile and reaction piles during the Static Load Test. The system included a Leica Absolute Tracker AT402 with laser reflectors installed on each of the five piles. In addition, a control network consisting of an additional six reflectors mounted on concrete pedestals and/or portions of the Massey Tunnel was installed around the pile test area to establish/maintain survey control of the laser tracking system during the test. The laser tracking system included its own data logger which was time synchronized with the main ADAS system so that the time-stamped laser tracking data acquired during the test could be aligned with the other instrumentation data acquired during the Static Load Test.

The laser tracking system was supplied and operated by Kodiak Measurement Services Inc. of Nelson, BC. The system had a precision of 0.1 mm in both vertical and horizontal directions which exceeded the minimum contract requirements.

The Lieca AT402 was calibrated prior to arrival at the site. The calibration records are provided in Appendix O.

11.4 Vibrating Wire Strain Gauges

A series of vibrating wire strain gauges was installed on the test pile and reaction piles to measure strain in the these piles prior to and during the Static Load Test. The method of installation of the below ground strain gauges on the test pile was provided in Section 7.1.6.1 above. In addition to the below ground strain gauges on the test pile, a single row of strain gauges was also provided at 0.3 m above the ground surface on each of the test and reaction piles.

Each of the above ground levels included vibrating wire strain gauges mounted directly on the pile sidewall(s) at 90 degree intervals (a total of four above ground strain gauges on each level for each of the five piles). The pre-test elevation of the above ground strain gauges was approximately +1.2 m on each of the piles. On the test pile only, seven levels of below ground strain gauges were provided, with six strain gauges mounted at 90 degrees apart on each level, except on Level 7 (elev. -9.1 m) where eight strain gauges were mounted. The below ground strain gauge levels were located at approximately elev. -64.1 m, -55.7 m, -47.7 m, -38.9 m, -30.5 m, -22.1 m , and -9.1 m. In total, 64 strain gauges were installed, including 44 below ground strain gauges on the test pile and 20 above ground strain gauges on both the test and reaction piles. The as-constructed locations of the strain gauge installations are presented on Figures 6, 7, 8.

The vibrating wire strain gauges were supplied by RST Instruments, model VWSG-A, with the following properties:

- Operating range: 0 to 3000 microstrain; and
- Precision: 1 microstrain.

The vibrating strain gauges were calibrated prior to delivery to the site. The calibration records are provided in Appendix O.

11.4.1 Strain Gauge Vibration and Impact Testing

Consistent with the RFP/contract requirements, Golder carried out physical testing of sample vibrating wire strain gauges prior to installation on any of the piles to determine whether the strain gauges could withstand the anticipated vibratory and impact forces that the strain gauges would be subject to during pile driving. Golder retained CascadeTek of Oregon USA, a specialist laboratory, to conduct the testing on the strain gauges consistent with the parameters established by Golder to best represent the anticipated loading conditions that the gauges would be subjected to during pile driving.

Golder used information obtained from previous projects, the anticipated subsurface conditions, and information supplied by the pile driving equipment manufacturers to determine the testing parameters for the sample strain gauges. The testing parameters included:

- Vibration Table (representing APE 200-6 vibratory hammer): 9.1 Hz, 30 mm amplitude, 1 hour duration; and
- Shock Table (representing D180-42 diesel hammer): 400 g to 450 g, 3 to 4 millisecond shock pulse, every two minutes for 4 hour duration.

The strain gauge functionality was monitored prior to, during and after all of the testing carried out at the laboratory and after shipped back to Golder’s Vancouver office. The testing results indicated that the strain functionality remained at 100% after being subject to the vibratory and shock loading. Upon arrival back to the Vancouver office, testing of the strain gauges indicated that they remained generally within 0.3% of their original settings, except for one gauge that remained within 2% of its original setting. These results demonstrated that the mechanical function, mounting/clamping function and cable function of the strain gauges was acceptable.

The detailed strain gauge testing report prepared by CascadeTek is presented Appendix P.

11.4.2 Strain Gauge Functionality

The functionality of each of the vibrating wire strain gauges was confirmed upon delivery to the project site and each strain gauge had full functionality prior to installation on any of the piles. The functionality of the below ground strain gauges was monitored throughout the test pile installation as well as the above ground strain gauges following installation.

Each of the above ground strain gauges remained fully functional during and following installation and through the Static Load Test. This was expected as the above ground strain gauges were installed after the piles were installed and, therefore, were not subject to any type of mechanical disturbance during pile driving or heat generated during welding of the below-ground protective cover plates.

Of the forty-four below ground strain gauges that were installed on the test pile, only thirty-two strain gauges remained functional immediately prior to the Static Load Test. The functionality of the below ground strain gauges was monitored at regular intervals during installation of the strain gauges and pile driving. It was observed during pile driving that many of the strain gauges became non-functional and/or could not be read with handheld monitoring equipment. The functionality/readability of the strain gauges appeared to be inversely proportional to their exposure to pile driving with the APE D180-42 diesel hammer (i.e. the strain gauges that were located in closest proximity to the pile driving equipment and exposed to the highest amount of impact repetitions appeared to lose functionality/readability first).

Following discovery of the first three non-functional strain gauges, and following thereafter up to and including the Static Load Test, Golder involved RST Instruments in testing/evaluation of the functioning and non-functioning gauges. The purpose of involving RST Instruments was to assess the strain gauge performance and prevent the loss of any additional gauges; however, the exact root cause of the strain gauge failures could not be determined. Different pieces of electronic equipment were utilized to read the strain gauges, with improved results. Further discussion on strain gauge functionality/readability is presented in Section 12.6.3 below.

Figures 6, 7 and 8 presents the vibrating wire strain gauge arrangement on the reaction and test piles along with presentation of the functioning/readable and non-functioning strain gauges at the start of the Static Load Test. The non-functioning/unreadable gauges at the start of the Static Load Test are highlighted in red on the figures.

The strain gauge readings obtained from each of the strain gauges from initial zero reading (prior to or immediately following welding of cover plate) to following the Static Load Test are presented in Appendix Q.

11.5 Telltales and LVDTs

11.5.1 Telltale Rods

Two 9.5 mm diameter, solid stainless steel, telltale rods were installed within 25 mm square steel tube housing(s) that were welded directly onto opposite sides of the test pile. The installation of the telltale threaded base and square housing is discussed in Section 7.1.6.2 above. The telltale rods were installed following installation of the test pile, cleanout and placement of the concrete plug. During installation of the telltale rods it was discovered that some obstructions were present within both (TT1 and TT2) of the telltale housings. The obstruction in TT1 was encountered at about 4 m above the target elevation and the obstruction in TT2 was encountered at about 5 m above the target elevation. The telltale rods were extracted and fine sand particles were observed on the end of the telltale rods. Based on the presence of sand on the end of the telltale rods and the resistance to telltale rod penetration through the obstruction(s), it was determined that the obstructions were loose sand that likely entered into the telltale housing during pile driving through unknown discontinuities in the telltale housing welds. It was proposed to clean out the telltale housings with pressurized water, but this was not permitted by MoTI. Instead, it was proposed and accepted to insert the telltale into the obstructions as far as possible and anchor the telltale rods using a male threaded tip installed at the end of the telltale rods (a female threaded tip was pre-machined into the end of the telltale rods originally).

The final installation of the telltale rods involved adding a short threaded tip onto the bottom of the telltale rods and setting the telltale rods as far into the sand obstruction as possible. For TT1, the tell-rod was successfully installed through the obstruction such that the bottom of the telltale rod was seated on the upper portion of the steel telltale housing (sudden hard resistance to penetration was encountered and the telltale rod could not penetrate any further). It was confirmed, by calculating the TT1 rod tip elevation that the telltale rod was at the upper portion of the telltale threaded base and would not be able to penetrate any further. For TT2, complete penetration through the sand obstruction could not be achieved and the telltale rod tip could only be installed to within approximately 2.5 m of the telltale rod threaded base. At the final installation depth, the TT2 rod could not be pulled out of the sand obstruction by hand, nor could the rod penetrate further into the sand without significant risk of damage to the rod. As such, it was decided to leave the TT2 rod short of its intended depth.

The final tip geodetic elevations for the telltale rods were as follows:

- TT1: elev. -65.05 m; and
- TT2: elev. -62.56 m.

Based on observations made during installation of the TT2 rod, it was determined that the TT2 rod could freely move between the rod tip (about 3.5 m above the pile tip) and the measuring point located above the ground surface (Linear Variable Differential Transducers, or LVDT) and, therefore, would be able to provide an accurate, direct measurement between these two points during the Static Load Test. Similarly, it was determined that the TT1 rod was able to provide an accurate, direct measurement between the top of the telltale threaded housing (located about 1.05 m above the pile tip) and the measuring point above the ground surface (LVDT).

11.5.2 Linear Variable Differential Transducers (LVDT)

Two Linear Variable Differential Transducers (LVDTs) were mounted directly onto the test pile above the top end of each telltale housing to precisely measure the telltale rod movements during the Static Load Test. Each of the LVDTs were clamped onto mounting blocks that were welded onto the side of the test pile. The mounting blocks also contained guide holes to stabilize the telltale rods and reduce jitter in the LVDT readings. The measuring end of the LVDTs was directly clamped to the telltale rods. The clamping mechanism allowed extension of the LVDT to the measuring range required by the RFP/contract specifications. The elevation of the LVDTs was be carefully selected to optimize the range of the LVDT measurement and physically fit onto the top of the test pit along with all the other instrumentation and structural elements. The LVDTs were both mounted at approximately elev. +1.8 m on the side of the test pile. The telltale and LDVT arrangement is shown in Figure 9.

The telltale and LVDT system was custom designed and supplied by RST Instruments and had the following properties:

- Operating range: 0 to 250 mm (extendable to 400 mm); and
- Precision: ± 0.1 mm.

Each of the LVDTs were calibrated prior to delivery to the site. The calibration records are provided in Appendix O.

The LVDTs were connected to the Automated Data Acquisition System (ADAS) which was configured to read the LVDTs at two second intervals.

11.6 Reference Datum

All elevations are referenced to geodetic datum.

In addition, and as mentioned above, a reference point located north of and outside of the zone of influence of the Static Load Test (at least 10 m away from any of the test and reaction piles) was established to provide stable, consistent vertical control for the Liquid Level system and Lieca AT402 laser tracking system. This reference point consisted of a 5 m long steel H-pile that was driven into the ground using the vibratory hammer that installed the pile guide frames and the test and reaction piles.

The location of reference H-pile is shown on Figures 3 and 5.

11.7 Automated Data Acquisition System

11.7.1 Design

The ADAS system was designed/supplied by RST Instruments Ltd. and was configured to read and record the following instruments at the required test intervals (discussed below) during the Static Load Test:

- Up to 64 Vibrating Wire Strain Gauges;
- 12 Load Cells;
- 2 Linear Variable Differential Transducers (LVDTs); and
- 2 Liquid Level Gauges.

All of the instrumentation identified above was supplied by RST Instruments to ensure compatibility between the data logging components and measuring instruments.

The ADAS system included the following components that were arranged as illustrated in Figure 12:

- RST Flexdaq Data Logger System including:
- RST Flexi-Mux Multiplexors;
- Campbell Scientific CR1000 data loggers;
- Campbell Scientific CR6 data loggers;
- Dell Latitude E6440 Laptop Computer;
- Uninterrupted Power Supplies; and
- 3 Display Monitors.

The technical details of the main components is presented the following sections.

11.7.2 Location

The location of the main ADAS system was in a portable construction office (Seacan) located approximately 25 m north of the test pile site. The location of the instrumentation Seacan is shown in Figure 2. Another, smaller component of the ADAS system was located at the jack operator location, immediately east of the test pile site.

11.7.3 Type, Make and Model

11.7.3.1 RST FLEXDAQ Data Logger System

The RST Instruments Flexdaq data logger system included two Campbell Scientific CR1000 and three Campbell Scientific CR6 data loggers and a series of RST Flexi-Mux Multiplexors.

The five data loggers were connected to a separate Dell Latitude E6430 laptop computer via a serial connection. Straight through serial cables were plugged into the data loggers' ports via a serial connection hub into the laptop computer's serial port. The baud rate for these communication ports was 115200 baud.

Additional details for the Campbell Scientific CR1000 and CR6 data loggers can be found at: https://www.campbellsci.com.

11.7.3.2 DELL Latitude E6430 Laptop Computer

The computer used to record, process and present the instrumentation data during the Static Load Test was a Dell Latitude E6430 laptop computer that was cleared of unnecessary software and memory use to ensure clean and efficient operation during the Static Load Test. The laptop computer was equipped with an Intel Core i5-2520M processor clocked at 2.5GHz, 8.0GB of RAM, and a 232GB hard drive. The laptop was be capable of displaying up to three monitors and this capability was utilized during the Static Load Test to present three different sets of the instrumentation data.

11.7.4 Recording

One of the CR1000 data loggers and all of the CR6 data loggers were programmed to reset and clock through the multiplexors connected to all of the strain gauges at regular 30 second timing intervals throughout the test. The other CR1000 data logger was programmed to reset and clock through the multiplexors connected to the load cells, LVDTs and liquid level gauge transducers at regular 2 second intervals throughout the test. The start of each recording interval was recorded (official timestamp) for each load increment to allow direct reference to each of the data sets acquired.

The CR1000 and CR6 data loggers recorded all instrument data on their own internal memory system. The CR1000 data loggers had a memory capacity of 4 MB, which is equivalent to approximately 8000 full sets of instrument readings. The CR6 data loggers had expandable memory capacity up to 32 GB (the size of the SD memory cards used during the test).

11.7.5 Software

Communications between the laptop computer and the data loggers was conducted through Loggernet Software Version 4.4. The Loggernet Software program was installed on the computer to acquire and copy the datasets from the each of the five data loggers. Data was automatically collected from the data logger units and appended to the .dat file(s) produced by the Loggernet software throughout the Static Load Test such that two individual databases were produce/updated in real time. The Loggernet copy of the instrumentation database was used by RTMC Pro, Version 3.2 (a separate software program) to process and display the instrumentation results in real time as the instrumentation data became available. Specifically for the Static Load Test, RTMC Pro was configured to display the following:

- Total Applied Load on Test Pile (Combined 12 Load Cells) vs. Vertical Deflection of Test Pile (Liquid Level Gauge(s)). This provided a continuous full load vs. deflection plot that could be monitored during the test;
- Instrument Reading vs. Time for each of the monitoring instruments (including individual strain gauges, load cells, liquid level transducers, and LVDTs). This data was displayed as individual plots on one graph; and
- Total Applied Load in real time (Combined 12 Load Cells). This provided the jacking operators with real time loading data to accurately apply/hold/release hydraulic pressure during the test.

Each of these data sets was displayed on individual monitors (5 monitors in total were used during the test).

All data obtained during the test was copied to our internal servers at the end of each test date. In addition, a copy of the Static Load Test data was provided to MoTI on a portable flash drive at the end of each test date.

Details on the Loggernet software can be found at: https://www.campbellsci.com/loggernet. Further detail on the Vista Data Vision software can be found at the following website: http://www.campbellsci.com/rtmcpro.

11.7.6 Power Supply and Backup

Electrical power to the ADAS. laser tracker, and laptop computer originated from the existing power grid at the site. The ADAS, laser tracker and laptop computer were connected to Uninterrupted Power Supply (UPS) units to provide continuous, short-term, power in case a power loss event was experienced. The UPS units were APC BE550G's, with an estimate runtime of 20 minutes if the original power supply became interrupted. The UPS was capable of providing power for a sufficient duration to allow connection of the ADAS system to a backup generator if a backup power supply was required. The backup generator was a Honda EU3000is portable generator and was maintained on standby during the Static Load Test.

The ADAS system and laptop computer also had built-in battery backup in case of power loss was experienced during the test.

11.8 Communication

Continuous communication between the Test Supervisor, Jack Operator and Laser Tracking System Operator was necessary during the Static Load Test to verify that the proper loading sequence was being followed at the designated holding times and that a complete data set was obtained at each loading interval. Motorola Radius CP200 two-way radios were during the Static Load Test for such communication purposes.

12.0 STATIC LOAD TEST

The Static Load Test was carried out in three individual phases on different days including an initial loading and unloading phase, a second loading and unloading phase, and a third loading and unloading phase. The following sections present the specific details of the overall Static Load Test.

12.1 Dates

The dates of the Static Load Test were as follows:

- Phase 1 Loading and Unloading: August 18, 2016;
- Phase 2 Loading and Unloading: August 19, 2016; and
- Phase 3 Loading and Unloading: August 31, 2016.

12.2 Weather Conditions

The weather conditions on each day of Static Load Testing were:

- August 18, 2016: Sunny; Temperature 16°C to 23°C;
- August 19, 2016: Sunny; Temperature 17°C to 26°C; and
- August 31, 2016: Cloudy/Occasional Rain; Temperature 15°C to 19°C.

All temperature measurements were obtained at Vancouver International Airport in Richmond, BC (approximately 10.0 km north-north-west from the Test Pile site).

12.3 Test Procedures

Each phase of the Static Load Test was carried out in general conformance with ASTM D1143-07 – Quick Test Method. The loading sequence and recording intervals were as reported in Section 10.5 above.

12.3.1 Phase 1 Loading and Unloading Sequence

The maximum test load of 45 MN could not be reached during the initial loading sequence due to excessive pile head deflection at lower test loads. As a result, the initial stage of the Static Load Test was carried out to approximately 300 mm of vertical pile head deflection, which was defined as the maximum pile head deflection criteria by MoTI for the Static Load Test. The test loads were generally applied by alternating sets of six jacks until leak issues were encountered with three of the jacks which took them out of service. Between test loads of 24 MN and 26 MN, the jack operator had difficulty applying a constant, evenly distributed load on the test pile, and it was decided to use one set of eight jacks to better distribute the test load evenly around the pile. During loading from

26 MN to 28 MN, it was decided to hold the test load when 280 mm pile deflection was achieved. As a result, the maximum test load achieved during the initial loading sequence was 26.2 MN and this test load was held for 15 minutes between approximately 280 mm and 297 mm pile head deflection. The unloading phase immediately followed the 15 minute hold period at 26.2 MN.

It is noted that a loud bang was heard during the early portions of the first unloading sequence. This appeared to coincide with the ring collar releasing from the side of the Test Pile.

12.3.2 Phase 2 Loading and Unloading Sequence

Similar to the Phase 1 loading sequence, the maximum test load of 45 MN could not be achieved due to excessive pile head deflection and meeting the maximum extension of the jacking (including shim) system. Since three jacks were not functional, it was decided to load the pile with two, alternating 4 jack systems up to 20 MN, then a combined 8 jack system for applied loads greater than 20 MN. The switch between and 4 jack and 8 jack system was made between application of the 16 MN and 20 MN test loads. A maximum test load of 26.7 MN was achieved during the second loading sequence at 413 mm pile head deflection. The Phase 2 loading sequence was terminated at this pile head deflection as the maximum stroke of the jacks was realized. The second unloading phase immediately followed a 6 minute hold period at 26.7 MN.

It is noted that some loud bangs were also heard during the late portions of the Phase 2 loading sequence. This appeared to coincide with the ring collar being dragged down with the Test Pile and becoming damaged (see photograph in Appendix S).

12.3.3 Phase 3 Loading and Unloading Sequence

The maximum test load of 45 MN could not be achieved during the Phase 3 loading sequence due to excessive pile head deflection and meeting the maximum extension of the jacking (including shim) system. All twelve hydraulic jacks were utilized (following repair to the jack seals) with the test loads applied by alternating sets of six jacks. Three hydraulic pumps were utilized concurrently to achieve the maximum test load of 29.1 MN during the third loading sequence at 257 mm pile head deflection. It should be noted that the vertical pile head measurements were zeroed at the start of the third loading sequence and, therefore, do not represent the full pile displacement for the three combined loading sequences. The Phase 3 loading sequence was terminated at this pile head deflection with the approval of MoTI. The unloading phase immediately followed a 15 minute hold period at 29.1 MN.

12.4 Special Adjustments

There weren't any special adjustments made during the Static Load Test except in preparation for the Phase 3 portion of the test where the following changes were incorporated

- The upper portion of the test pile was completely filled with concrete;
- Additional shims and spacer plates were required to extend the range of the jacking system. A layer of twelve – 350 mm high, 200 mm diameter solid steel shims was provided immediately above the load cells at the exact same spacing/location as the load cells. An additional 2.20 m diameter, 25 mm thick spacer plate was provided between the new shim layer and jack layer. All the jacks, shims and load cells were placed at 30 degree spacing and 910 mm diameter offset from the center of the pile;
- Other adjustments to the instrumentation, including liquid level system, were necessary to extend the measuring range of the instruments; and
- The surficial materials located immediately surrounding the test pile were locally excavated to approximately 0.4 m below the Phase 1 and Phase 2 loading sequence ground elevation to allow the side-mounted instrumentation to extend below the surrounding ground surface.

12.5 Data Recording

The instrumentation data was recorded continuously throughout the Static Load Test as described in Section 11 above. In summary, the instrumentation data was recorded at the following frequency:

- Test Pile and Reaction Pile Strain Gauges: Every 30 seconds;
- Test Pile Liquid Level, Load Cell and Telltale/LVDT Systems: Every 2 seconds; and
- Laser Tracking System: 30s, 1 min, 2 min, 4 min, 8 min, 10 min, and 15 min intervals at for each of the initial and third load stages; at 10 min intervals for the second load stage; at 5 min intervals for each unloading stage and zero load stage (for 30 min).

The data was recorded instantly on each of the five data loggers and immediately backed up and plotted by the Dell laptop computer. The data was compiled and a copy of the data was provided to MoTI on a portable flash drive at the end of each day of testing.

12.6 Test Results

The instrumentation data obtained during the three phases of the Static Load Test were compiled and processed following completion of the Static Load Test. The raw data set obtained from the geotechnical instrumentation has been provided to MoTI in electronic format. As per MoTI requirements, the instrumentation data for required time interval at each load stage have been obtained and tabulated. Various summary plots of the instrumentation data have also been produced as per MoTI requirements.

It is noted that the instrumentation data presented herein summarizes the MoTI specified data sets (a minimum contract requirement) and does not include all of the data provided to MoTI in the electronic data files.

12.6.1 Pile Load/Deflection

The pile load vs. pile vertical deflection plots were plotted real-time (30 second intervals) while each stage of the Static Load Test was carried out. For reporting purposes, the pile load vs. deflection tables and plots include the load and deflection measurements obtained at the end of each holding period. The holding periods include:

- At the end of 15 minutes for Phase 1 loading and Phase 3 loading sequences;
- At the end of 5 minutes for Phase 1, Phase 2 and Phase 3 unloading sequences;
- At the end of 10 minutes for the Phase 2 loading sequence; and
- At the end of 30 minutes when each pile is unloaded at the end of each of the three test phases (zero load).

The tables and plots presenting the pile load vs. deflection data are presented in Appendix R.

12.6.2 Telltale Readings/Load

Telltale/LVDT measurement vs. time plots were plotted real-time (30 second intervals) while each stage of the Static Load Test was carried out. For reporting purposes, pile load vs. telltale measurement tables and plots included the load and telltale/LVDT measurements obtained at the end of each holding period, including:

- At the end of 15 minutes for Phase 1 loading and Phase 3 loading sequences;
- At the end of 5 minutes for Phase 1, Phase 2 and Phase 3 unloading sequences;
- At the end of 10 minute for the Phase 2 loading sequence; and
- At the end of 30 minutes when each pile is unloaded at the end of each of the three test phases (zero load).

The tables and plots presenting the pile load vs. telltale (pile compression) measurements are presented in Appendix R.

In general, when comparing the telltale data obtained from TT1 and TT2, there is consistent agreement between the two telltale readings, with only slight divergence due to the telltale tips being located at different elevations. The only significant difference observed between the two telltale readings was when the pile was being unevenly loaded due to hydraulic jack leak(s) and removal/replacement during the Phase 1 load sequence. The observed difference in the telltale readings is an expected response and was confirmed by the above-ground strain gauge readings which also diverged when the test pile was unevenly loaded. Based on review of the telltale data obtained, it can be concluded that both telltales were fully functional as intended.

12.6.3 Strain Gauge Readings/Time

As discussed in Section 11.4.2 above, the overall functionality/readability of the below-ground strain gauges was reduced during pile driving. At the end of pile driving and PDA testing, only 17 of the 44 below-ground strain gauges could be read with the available hand-held monitoring equipment. The below-ground strain gauges were connected to the ADAS approximately 2-3 weeks prior to the Static Load Test to permit near-continuous reading of the strain gauges and analysis of the results. Based on the analysis of the below-ground strain gauge readings obtained during this recording, the ADAS was modified to maximize the readability of the below-grade strain gauges. A brief discussion about the usability of the strain gauge data is provided below.

12.6.3.1 Strain Gauge Data Obtained Prior to the Static Load Test

Periodic readings of the strain gauges were obtained using hand held monitoring equipment and manually recorded during installation of the Test Pile. The readings were typically obtained at the following milestones:

- Prior to strain gauge installation onto each Test Pile segment;
- After strain gauge installation on each Test Pile segment;
- After each Test Pile segment was stacked for splice welding;
- Before and after cover plate welding;
- Before and after PDA testing;
- Before and after soil cleanout;
- Before/during/after concreting; and
- Before connection to the ADAS.

The below ground strain gauges that were exposed to most significant pile driving activity (i.e. the deeper strain gauges) and that were mounted closest to the pile driving equipment (and possibly exposed to higher temperature during welding of the cover plates) had diminished readability as pile driving continued. Many of the deeper strain gauges (gauges that were exposed to the highest repeated pile driving impacts) generally became unreadable during pile driving. Following installation of the Test Pile, only 17 of the 44 below ground strain gauges were readable using the hand held monitoring equipment. A summary of the below ground strain gauges that were readable using the hand held equipment following pile driving include the following:

Table 17: Readable Below Ground Strain Gauges Following Pile Driving

SGTPL2-E
SGTPL3-C
SGTPL5-A
SGTPL5-B
SGTPL5-F
SGTPL6-A
SGTPL6-B
SGTPL6-C
SGTPL6-D
SGTPL6-E
SGTPL6-F
SGTPL7-A
SGTPL7-B
SGTPL7-E
SGTPL7-F
SGTPL7-G
SGTPL7-H

Strain gauge reading vs. time plots and summary tables are presented in Appendix Q. The gaps shown in the some of the strain gauge reading plots and the 'ND' value in the tables indicate where strain gauge readings could not be obtained.

12.6.3.2 Strain Gauge Data Obtained During the Static Load Test

As discussed above, strain gauges readings were obtained every thirty seconds during each day of Static Load Testing. Specialized dataloggers were used in the ADAS that could adjust the sensitivity and readable range of the dataloggers to maximize the ability to read the strain gauges. The reconfigured ADAS was able to obtain usable readings from 32 of the 44 below ground strain gauges during the Static Load Test. In addition, 'clean' readings were successfully obtained from all 20 of the above ground strain gauges during the Static Load Test.

The majority of the usable readings obtained from the below ground strain gauges was relatively 'clean'; however, several of the readable strain gauges provided 'noisy' readings that followed a similar trend line/curve to the trend lines/curves produced by adjacent 'clean' strain gauge readings. The remaining below ground strain gauges were unreadable.

A summary of the strain gauge readability/usability is presented in Table 18 below.

Table 18: Strain Gauge Data Usability

‘Clean’ Strain Gauge Data	‘Noisy ‘Strain Gauge Data	Un-Usable/No Strain Gauge Data
SGTPL1-A	SGTPL1-D	SGTPL1-B
SGTPL2-E	SGTPL1-E	SGTPL1-C
SGTPL2-F	SGTPL1-F	SGTPL2-A
SGTPL3-C	SGTPL2-D	SGTPL2-B
SGTPL4-B	SGTPL2-C	SGTPL3-D
SGTPL4-C	SGTPL3-A	SGTPL3-E
SGTPL4-E	SGTPL3-B	SGTPL3-F
SGTPL4-F	SGTPL5-D	SGTPL4-A
SGTPL5-A	SGTPL5-E	SGTPL4-D
SGTPL5-B		SGTPL5-C
SGTPL5-F		SGTPL7-C
SGTPL6-A		SGTPL7-D
SGTPL6-B		
SGTPL6-C		
SGTPL6-D		
SGTPL6-E		
SGTPL6-F		
SGTPL7-A		
SGTPL7-B		
SGTPL7-E		
SGTPL7-F		
SGTPL7-G		
SGTPL7-H		
SGTPL8-A		
SGTPL8-B		
SGTPL8-C		
SGTPL8-D		
SGRP1-A		
SGRP1-B		
SGRP1-C		
SGRP1-D		
SGRP2-A		
SGRP2-B		
SGRP2-C		
SGRP2-D		
SGRP3-A		
SGRP3-B		
SGRP3-C		
SGRP3-D		
SGRP4-A		
SGRP4-B		
SGRP4-C		
SGRP4-D		

The readings obtained from the ‘noisy’ strain gauges was averaged and plotted to establish average strain readings for each of the ‘noisy’ gauges. The strain gauge reading vs. time plots and data tables presented in Appendix Q include both the ‘clean’ strain gauge readings and the calculated average strain readings for the ‘noisy’ gauges listed above. The average readings, along with the ‘clean’ strain gauge readings, were used to estimate the load distribution along the pile shaft during Static Load Testing as discussed below.

12.6.4 Concrete Plug Observations

Following completion of Phase 1 and Phase 2 of the Static Load Test, holes were cut through the pile sidewall and the lower Load Plate to measure the location of the top of the concrete plug. Upon exposure of the interior the Test Pile, standing water was observed above the concrete plug. The following measurements were made:

Table 19: Top of Concrete Plug Measurements

Measurement Location	Measured Distance From Top of Test Pile Cut Off Prior to Static Load Test (mm)	Measured Distance From Top of Test Pile Cut Off Following Phases 1 and 2 of Static Load Test (mm)	Measured Water Depth From Center of Pile
North Side of Test Pile	3000	3004	0.05 m
West Side of Test Pile	2931	2932	
South Side of Test Pile	2964	2956	
East Side of Test Pile	2961	2981	

Water was also observed seeping from the access holes cut in the upper Load Plate for concreting purposes during the latter portions of the Phase 3 test.

12.6.5 Estimated Load/Pile Depth

The load distribution along the pile shaft was estimated using the usable strain gauge data obtained prior to and during the Static Load Test as discussed above. The strain gauge data obtained at each level along the pile shaft was assessed for usability, averaged across the pile area at each strain gauge level and converted to average load at each level. The average estimated load at each level was plotted along the length of the pile for the following conditions:

- Loads at the end of pile driving;
- Loads before and after the restrike PDA test;
- Loads immediately before and after the 20 m cleanout of the pile;
- Loads after concrete infilling;
- Loads immediately before the start of the Static Load Test; and
- For each step of the applied load during all three loading and unloading phases.

The estimated load distribution for each of the above conditions has been tabulated and plotted in Appendix R.

It is noted that the estimation of the load distribution along the pile shaft was produced at the request of MoTI to address specific preliminary design purposes and involved a significant amount of interpretation of the strain gauge data and engineering judgement. It is recommended that other parties using the Static Load Test data carry out their own interpretation of the data to determine their own estimate load distribution along the pile shaft.

The assumptions made to develop the estimated load distribution along the pile shaft include the following:

- The zero readings for the strain gauges were generally assumed to be when each individual pile segment was stacked and the strain gauge cover plate was not yet installed. The only exceptions to this was for Level 1 strain gauges where the zero was taken after the cover plate was welded and pile segment was stacked and for Level 2 gauges where the zero was taken after the strain gauges were mounted and the cover plates were welded;
- The strain gauges used to estimate loads prior to the Static Load Test include SGTPL1-A, SGTPL1-F, SGTPL2-E, SGTPL2-F, SGTPL5-B, SGTPL5-F, SGTPL6-C, SGTPL6-E, SGTPL7-A, SGTPL7-H, SGTPL8-A and SGTPL8-C. The pre - Static Load Test strain gauge readings obtained from Level 3 and Level 4 gauges were considered suspect and therefore the loads at these levels were estimated based on the strain gauge readings obtained on the adjacent levels. Where possible, strain gauge readings from opposing sides of the Test Pile were used and averaged for conversion to load prior to the Static Load Test;
- 'Noisy' strain gauge readings were averaged as discussed in Section 12.6.3.2 above and used in the load estimation;
- Another set of zero load readings were obtained prior to the start of the first and second phases of the Static Load Test on August 17, 2016 at 4:20 pm. This was the time prior to the test when applied loads to the Test Pile due to thermal shrinkage/swelling of the test frame and reaction piles was minimal. The actual load on the pile at this zero reading was 0.26 MN;
- A third set of zero load readings were obtained prior to the start of the third phase of the Static Load Test on August 31, 2015 at 7:02 pm. There were no thermal loading effects on the Test Pile at the start of the third phase of testing;
- To estimate the loads at each strain gauge level during the Static Load Test, the usable strain gauge readings (including 'noisy' gauges) at the each level were averaged. One set of opposing strain gauges were used on Level 1 (SGTPL1-A, SGTPL1-F), Level 2 (SGTPL2-C, SGTPL2-E), Level 3 (SGTPL3-A, SGTPL3-C), Level 4 (SGTPL4-C, SGTPL4-E), and Level 5 (SGTPL5-B, SGTPL5-F) and two sets of opposing strain gauges were used on Level 6 (SGTPL6-B, SGTPL6-D, SGTPL6-E, SGTPL6-F), Level 7 (SGTPL7-A, SGTPL7-B, SGTPL7-F, SGTPL7-H), and Level 8 (SGTPL8-A, SGTPL8-B, SGTPL8-C, SGTPL8-D);
- The material and sectional properties for the steel pipe section of the Test Pile were assumed to be E = 194 GPa and Area = 0.1901 m^2;
- The material and sectional properties for the composite (steel and concrete) section of the Test Pile were estimated using the Tangent Modulus method (ref. *"Tangent Modulus of Piles Determined From Strain Data"*, B. H. Fellenius, 1989). The Tangent Modulus equation used in the estimation was $M_t = A*\varepsilon + B$; where A = -0.0111 GPa/µƐ and B = 17.548 GPa for Level 7 strain gauges and A = -0.0005 GPa/µƐ and B = 12.2 GPa for the Level 8 Strain Gauges during the Phase 3 testing sequence. The pile area was assumed to be 3.1415 m^2 for both Level 7 and 8 cross sections.

13.0 CLOSURE

As identified above, this report addresses the work carried out for the Static Load Pile Test program as defined by the RFP/contract and specifically does not include site characterization, interpretation of data (except as defined herein), nor engineering assessment or recommendations. Any interpretation presented herein was intended for use of Golder and MoTI only. Other parties using the factual information obtained during the Static Load Test should carry out their own interpretation using their own assumptions, analysis and engineering judgement.

We trust the information presented in this geotechnical report is sufficient for your current requirements. Please do not hesitate to contact us should you have any questions or require further clarification regarding the information presented herein.

GOLDER ASSOCIATES LTD.

PROFESSIONAL PROVINCE OF P.B. BAKKER 27593 BRITISH COLUMBIA ENGINEER Oct 18/16

Pierce Bakker, PEng
Associate, Senior Geotechnical Engineer

M. (Yogi) Yogendrakumar, PhD, PEng
Principal, Senior Geotechnical Engineer

PBB/MY/nnv

o:\final\2015\3 proj\1533603 bcmoti_hwy 99 gmt geotech investigation\1533603-027-r-rev1\1533603-027-r-rev1-masseytunneltestpile 18oct_16.docx

IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

Standard of Care: Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practising under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

Basis and Use of the Report: This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client. The factual data, interpretations and recommendations pertain to a specific phase and site location of the project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. Any other use of this report by others is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product, and Golder authorizes only the Client and Approved Users to make copies of the report but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes and the amount of information collected at the site, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions and information collected may affect their work, including but not limited to proposed design implications, construction techniques, schedule, safety and equipment capabilities.

Soil, Rock and Groundwater Conditions: Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, *etc.*) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

Follow-Up and Construction Services: All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

Changed Conditions and Drainage: Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.