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Full-Scale Load Testing of 57-Year Old Raymond Piles for Foundation Re-use

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FULL-SCALE LOAD TESTING OF 57-YEAR OLD RAYMOND PILES FOR FOUNDATION RE-USE

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ABSTRACT

The proposed reconstruction of a demolished coke battery superstructure of a steel mill to a level higher than originally constructed in 1952 required the evaluation of the geotechnical resistance and settlement characteristics of its existing foundation piles. Except for design drawings showing the layout of the substructure elements and the borehole logs, there were neither as-built drawings nor any records available about the design and construction of the piles. A total of nine (9) axial compressive load tests were initially undertaken. Two (2) 19 ft long piles were initially tested under the pusher tracks, and a further seven (7) piles of varying lengths under the foundation of the coke batteries. These piles were assumed to have been installed into hard clay till underlying the site. The load tests showed that the mandrel driven outer steel casing and concrete in-filled piles could accommodate a load of 90 tons with minimal settlement. Cylindrical cores of the pile concrete taken following the load testing provided compressive strengths varying from 46 to as much as 62 MPa. This case study provides details of the excavation and dewatering issues, load test set-up, and the difficulties encountered in assessing and testing old foundations for re-use in a confined underground environment. It is hoped that this case study and its findings will encourage the proper assessment and evaluation of existing foundations as this could result in considerable savings in superstructure revitalization, emphasize the need to maintain design and construction records, and to instrument and monitor important foundations for long term reuse.

INTRODUCTION

Infrastructure improvement works at the Essar Steel Mill in Sault St Marie, Ontario required the assessment and evaluation of existing piles supporting the demolished No. 6 Coke Battery. The ground level location of the demolished No. 6 Coke Battery within the steel mill complex, and non-operational coke ovens of an adjacent Coke Battery is shown in Figs. 1 and 2, respectively.

Fig.1. Existing Surface of Oven Pad - No. 6 Coke Battery

Fig.2. Adjacent Non-Operational Coke Ovens

Openings shown in Fig.1 on the surface of oven pad are locations for flue gas pipes leading to the battery ovens.

The purpose of this assessment and evaluation was to determine whether the existing piles would be capable of supporting a new coke battery which would result in the design load on an existing pile being increased from 45 tons to 55-60 tons. This work associated with this task required locating the existing piles, visually observing their condition, where feasible, and conducting axial compressive load tests on a few of these piles.

PILES TESTED

Static axial compressive load tests were undertaken on a total of nine (9) piles within the existing No.6 Coke Battery complex. Pile load testing on two (2) of four (4) piles, initially identified for testing, was undertaken on April 14 and 15, 2009. The two other piles could not be located as a result of excavation difficulties encountered during April 17 and 24 resulting in the temporary suspension of the operations.

Similar load tests were undertaken on the seven (7) remaining piles. Four (4) of these were tested on July 13 and 14, and the remaining three (3) on August 11, 2009.

The piles tested on April 14 and 15 were located under the "pusher tracks" where the coke pushers travel along the front of the coke batteries. The remaining seven (7) piles were located under the coke batteries. Fig. 3 shows the approximate locations of the piles that were tested while Section AA in Fig.4 shows the pile top elevation below the pusher tracks and Section BB the pile top elevations below the coke ovens.

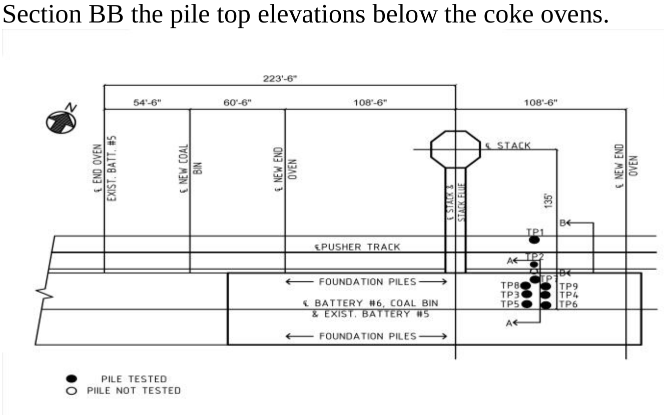

Fig.3. Approximate Locations of Piles Tested

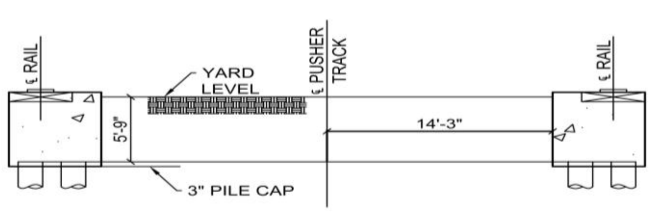

Fig.4. Section AA–Pile Top Elevation of Pusher Tracks

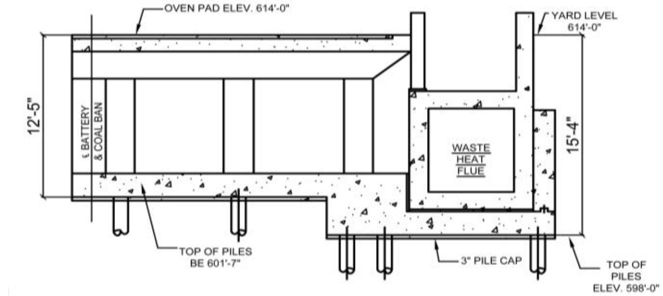

Fig.5. Section BB – Pile Top Elevations under Coke Ovens

BACKGROUND INFORMATION ON EXISTING PILES

According to the available historic project drawings for the No. 6 Coke Battery, this infrastructure was designed and constructed in 1952. As indicated on the drawings, the design was done by Koppers Company Inc. Engineering and Construction Division, Pittsburg P.A.

The "Notes" on the drawings stated that the Yard Level (Finished Ground Level) was El 614 and that the piles were to be steel cased concrete piles with a minimum point (toe) diameter of 12 inches and minimum butt (top) diameter of 14 inches with a minimum spacing between piles of 3 ft centre to centre. The piles were to be driven to a resistance of 8 blows per inch for the last 3 inches of penetration with a No.1 Vulcan Hammer or equivalent. A 1:2:4 concrete mix by volume was specified to achieve 28-day compressive strength of 2500 p.s.i. The maximum pile load was stated to be 45 tons which included loads from future 1800 ton coal bin.

The project drawings were not provided until after the pusher track piles were tested. Prior to that time, the piles were known to be capable of taking 45 tons. No factor of safety (FOS) was mentioned, but may have likely been a value of 3 or 4.

Without the knowledge of the type of driven cast-in-place pile used the information led to an investigation of the pile type used since typically cast-in-place piles in today's practice are not normally driven.

In today's geotechnical engineering practice, cast in-place piles are typically known to be constructed by drilling a pile hole with a motorized auger and constructing the pile by free falling and/or vibrating concrete into the formed hole. The formed pile often has a rebar cage included as reinforcement. Drill casing is used if sloughing subsurface conditions are anticipated. This casing can be used as a temporary measure and withdrawn after the concrete is poured or used as a permanent measure.

Other typical common names for this pile type used in today's practice are bored piles, drilled shafts and caisson piles. More recently, Auger Cast-in-Place and Continuous Flight Auger (CFA) piles have been added to this category of piling.

From a review of the information and the literature on piling, the pile that was used appeared to be a “driven and cast-in-place” pile which can be constructed by driving a steel tube or precast concrete shell which remains in the ground or with the steel tube or precast concrete shell withdrawn (Tomlinson, 1986; Seeyle 1960).

After further review of the literature, it was believed that the pile type used was the Raymond Constant section pile. However, on-site opinions on the pile type anticipated before any excavation was undertaken varied from pile construction with an outer steel shell being driven and withdrawn, to pile construction with the steel shell being left in place.

INFORMATION ON SUBSURFACE CONDITIONS

Subsurface conditions relevant to the No.6 Coke Battery site were provided by two (2) boreholes logs, borehole (BH) 293 and BH 295 as indicated on Fig.1. These were shown on the drawing sheet titled “Ovens Battery Foundations Piling Bid Sheet For 45 ton Piles”. The borehole logs were illustrated as plots of Standard Penetration Resistance (SPT) values versus depth and with stratigraphic descriptions with elevation and depth. Borehole 295 was the most relevant as this was in close proximity of the piles tested.

In general, in the case of BH 295 the stratigraphic profile consisted of 14ft of black slag overlying red sandy till whereas in BH 293 the thickness of black slag was 11 ft with a 4 ft layer of grey brown sand and grey brown sand and silt between the bottom of the slag and underlying red sandy till. As noted from the borehole logs, both boreholes were terminated in the red sandy till at an average depth of about 21 ft below the ground level. The ground level at the borehole locations at the time of the investigation showed a difference of about 2ft between BH 293 (El 611) and BH 295 (El 609) with the higher ground at BH 293 location.

The SPT blow count in the slag layer averaged about 10 in BH 293 and 20 in BH 295 signifying the layer to be in a loose to compact state. The SPT blow counts recorded at the end of each borehole (El 589.6 in BH 293) and (El 586.7 in BH 295) varied from 16 in BH 293 to 10 in BH 295. At the end of each borehole a dynamic cone penetration test was undertaken which showed increasing blow counts from 20 in BH 293 to greater than 100 blows per foot at El 585 a depth of 5 ft below the bottom of the borehole, and from 10 to greater than 100 at El 582 in BH 295.

Based on the low blow counts at the bottom of the borehole it is conceivable that the outer shell casing was driven into the red sandy till to a depth of at least 21 ft below the existing ground at the time of pile installation in 1952.

Groundwater level was recorded at 3‘ 9“ below ground level in BH 295 and at 5’7” below ground at the BH 293 location. In relation to elevations the groundwater was the same level in the two boreholes.

PROJECT INITIATION AND SAFETY MEETING

A project initiation meeting was held on April 7, 2009 at the Algoma Energy Co-Generation Site Office located within the Essar Algoma Steel Mill complex. At this meeting, the number of piles to be tested was discussed and locations tentatively identified.

There was some discussion on the difficulties that may be encountered in locating the piles since there was an understanding that the ground water levels were high within the site. It was agreed that the field work would commence following the meeting since the backhoe had been mobilized on site.

This meeting was followed by a safety orientation meeting since this was mandatory before any work could start on site. Prior to the start of the safety orientation there was some further discussion with the Safety Officer who advised that it would be difficult to excavate and find the piles at the locations that were initially intended. It was suggested and agreed upon that piles along the north side of the pusher track would be better to attempt to locate and test.

FIELDWORK FOR PILE No.1 UNDER PUSHER TRACK

A field review was undertaken following the safety meeting and the location for excavating for the first pile identified on the ground. The backhoe excavator was moved to site on April 7 but no work was started as intended since utility clearances were incomplete until around the end of the scheduled working day for the Contractor. Excavation was rescheduled to start on April 8.

The excavation process was slow as ground water was encountered around 3ft below the existing ground as had been advised. In order to be able to dig deeper and counteract the backfill sloughing a sump pump had to be used continuously as the excavation progressed. After a few hours of excavating and searching for the pile location, the pile was finally located.

The exposed section of pile above the ground water approximately 2ft appeared to be in good condition from visual observation. The outside of the pile appeared to be ribbed but this was later confirmed after excavation of pile No. 2 to be part of a corrugated/ribbed metal shell. The operation ceased for the day as the pile was identified found toward the end of the working day. Figure 6 shows the excavation in progress and Figure 7 the ground conditions encountered as the excavation progressed to locate and expose the pile under the pusher tracks.

Fig.6. Excavation to Expose Pile under Pusher Track

Fig.7. Located Pile under North Side of Pusher Track

PILE LOAD TEST – PILE No 1

Preparation for the pile load testing was discussed with the excavation contractor and it was agreed that on site available steel plates will be used to provide the dead load reaction. Since there was significant preparation to be made for field setup and testing, the load test was scheduled for April 14th 2009. The test setup developed is shown in Fig. 8. The dial gauges were anchored outside of the pile but with the stems resting on C-channels attached to the exterior of the pile.

In preparing the pile for testing, the pile capping beam shown in Fig. 5 was broken by a hydraulic rock breaker attached to the backhoe to expose the top of the pile. The diameter at the top of the pile was measured as 20 inches which was about 100 mm larger than that specified on the 1952 drawings.

Following breaking the capping beam from the pile head, the damaged pile head was prepared for the placement of the base plate and jack by the application of cement grout on the top of the pile to create a level surface.

Fig. 8. Test Set Up For Pile Load Test

The pile load test was started around 10.30 am on April 14 and was conducted as shown in the Photo above. The loads were applied through the hand pump that was used to transmit the hydraulic pressure to activate the jack. A digital readout gauge from the load cell was used to record the load in pounds or kilograms provided by the jack through a load cell located above the jack and bearing on the H-beam.

The ASTM Quick Load Test method outlined under ASTM D 1143-81 “Piles under Static Axial Compressive Load” was used in carrying out the test. The full load was removed from the pile in decrements after the test was taken to greater than twice the desired design load of 60 tons.

Three (3) load and unloading tests were undertaken on the same pile. In the first test or loading cycle, the test was taken to 100 tons since the readout gauge was starting to show fluctuations in the load indicating that the jack could not hold higher loads.

Malfunctioning of the jack was again noted when the load was being released from the hand operated hydraulic pressure pump. This unit would not allow the load to be released from the jack despite the hydraulic pressure valve being placed in the deflated position. Hydraulic fluid was noted to be leaking from the unit and as a result the unloading portion of the test could not be undertaken.

The defective hand operated hydraulic pressure pump was exchanged for a functional one from the equipment supplier and after the lunch period a second test was conducted on the same pile. During this test, the hydraulic system functioned properly, however the increment of loads intended to be applied in pounds was applied in equivalent kilograms instead, and hence a load increment thought to be applied in pounds was approximately 2.2 times the originally intended load. This load test was taken to 263,000 lbs before the error was recognized. The unloading test was undertaken in kilograms. A third test was undertaken after the issue with the last test was rectified.

FIELDWORK FOR PILE No.2 UNDER PUSHER TRACK

Excavation to locate and expose Pile No. 2 began on April 15. This pile was exposed the same day and was in alignment with Pile No. 1 and under the south beam of the pusher track. The location of this pile is shown in Fig 3. The excavation was undertaken in almost completely dry conditions since groundwater was drawdown by pumping from one compartment to the other within the adjacent Coke Battery Foundation. Quite noticeable was the steel shell/casing on the outside of the pile (Fig. 9). This observation confirmed that a casing was driven and left in place and confirmed that the pile was of the Raymond Type which was popular in those days. Since the entire pile was not exposed there was no way to assess whether the pile was tapered. The diameter at the top of the pile was around 20 inches.

Fig.9. Exposed Pile Showing Thin Steel Casing

FIELDWORK FOR PILES BELOW COKE BATTERIES

Selection of the seven (7) additional piles for load testing, and organization of the demolition and dewatering work involved in exposing the piles were undertaken under the overall direction of the Project Manager, Co-Generation Project.

For access to these seven (7) additional piles, portions of the concrete top slab and column supports for the battery ovens had to be demolished since these piles were located between 13 and 15 ft below the surface of the top slab, which was approximately coincident with the surrounding ground level (Fig.1). The demolition work was undertaken with a backhoe equipped with a rock breaker. During the demolition operation continuous site dewatering was undertaken.

The piles in the two-pile and three-pile groups associated with the coke battery foundations were prepared for testing as individual piles by demolishing the columns they supported and breaking the beams interconnecting the pile tops. The dead load reaction consisted of nine (9) slabs weighing about 252 tons. This load was increased to 276 tons by the addition of an additional slab after the testing of the first two-pile group was undertaken. This additional load was required to avoid lift-off of the weights observed when increasing taking the jack load to between 150 and 200 tons.

Two H-Sections were welded to the underside of the lowermost slab at about the centre spacing of the piles to allow the transfer of the jack loads to the reaction weights thereby allowing for testing two piles in a single set-up of the reaction weights. A load of about twice the anticipated failure load was aimed at for the total reaction weight. Figures 7 and 8 show the reaction weights and set-up for applying the load to the piles, respectively.

Fig.10. Reaction Weights- Steel Slabs

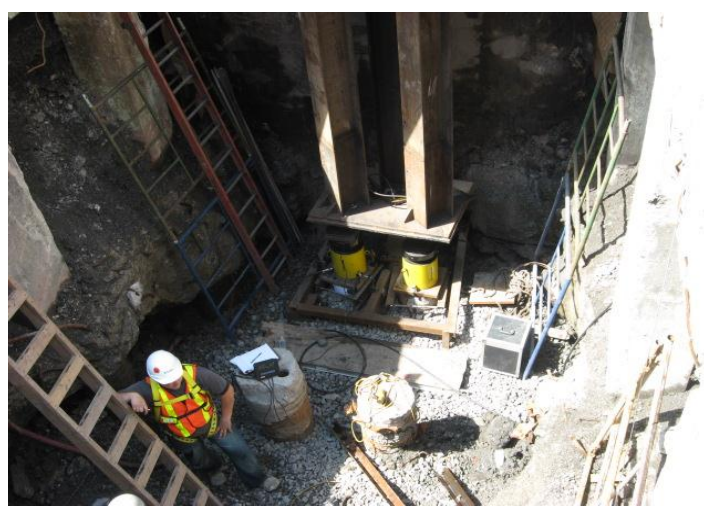

Fig.11 Pile Load Testing Set-up

PILE LOAD TEST RESULTS (Piles under Pusher Tracks)

Testpile No.1

Three load and unloading tests were carried out on Testpile No.1. The load-deflection graphs for these tests are shown in Fig. 12 for the loading potion only. In general, very small deflections were recorded on the loading cycle. The deflection at the maximum applied load of 200,000 lbs in Test 1 was 0.18”, while for the two other tests the deflections were 0.088” for Test 2 (Fig.13) under a maximum load of 263,000 lbs, and 0.068” under a maximum load of 180,000 lbs for Test 3 (Fig.14). The deflection obtained in Tests 2 and 3 being smaller than those recorded in Tests 1 could have resulted from cycling the pile since cyclic loads less than the failure load tend to result in increased pile stiffness and hence smaller deflections, generally.

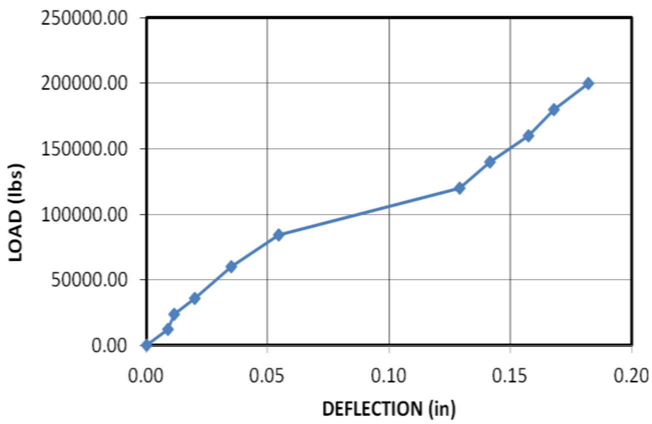

Fig.12. Testpile Graph for Pile No.1, Test No. 1

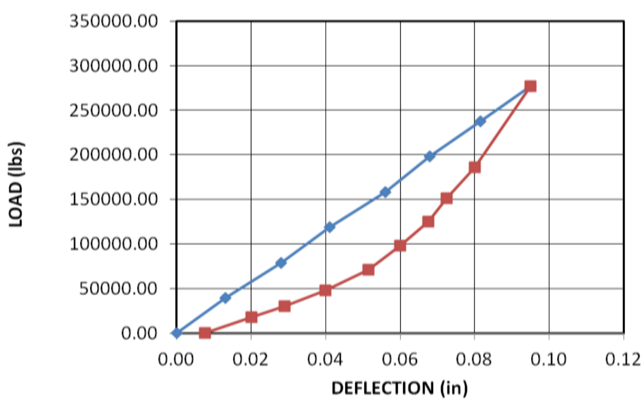

Fig.13. Testpile Graph for Pile No.1, Test No. 2

However, this was not the case; rather there was a problem in the correct reading of the dial gauges due to the tight working space and the fact that this was the first test undertaken. This error was noted after a load of 84,000 lbs had been applied.

Corrections were made to the readings recorded based on observations made by the dial gauge reader. As can be seen from the Test No. 1 (Fig.12) graph in comparison with the graphs of the other tests, there is an obvious discrepancy. Nonetheless, the overall deflections are small and hence of no consequence especially since the deflections recorded in the other tests were comparable

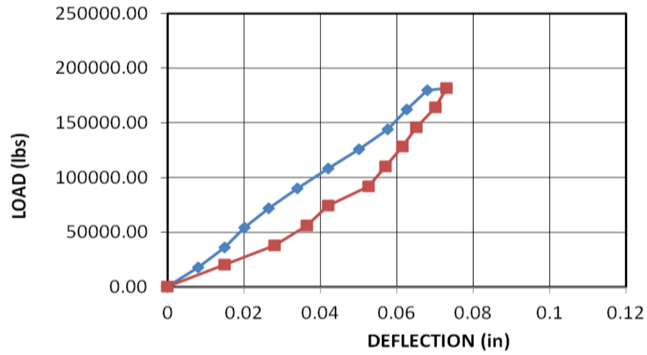

Fig.14. Testpile Graph for Pile No.1, Test No. 3

Conducting the two other loading tests on the same pile was not intended initially but was a consequence of this error and in the case of Pile Test 3, this was done to recheck with smaller applied load increments rather than the larger increments used as a result of the digital readout being set to read in kilograms rather than in pounds.

Testpile No. 2

The results of the load – deflection graphs of loading and unloading cycles are provided in Fig. 13. The deflection under the maximum load of 250,000lbs was approximately 0.09 in. The unloading curve returned almost to zero indicating that the loading produced essentially elastic compression of the concrete. The core compressive strength of the pile concrete was 64 MPa, which is unusually large, and almost 1.5 times the corresponding strength of the core from Pile No 1. Being stiffer, the deflection on loading was slighter smaller than those obtained for Tests 2 and 3 of Testpile No. 1.

The loading curve for Testpile 2 is shown in Fig. 15 along with the loading curves from Testpile No.1. The greater stiffness and smaller deflections can be readily seen when compared with the other tests. Testpile No.2 being not too far away from Testpile No.1 would be expected to have encountered similar ground conditions. Very likely, this pile would have attained a toe elevation of at least 21 ft below ground level.

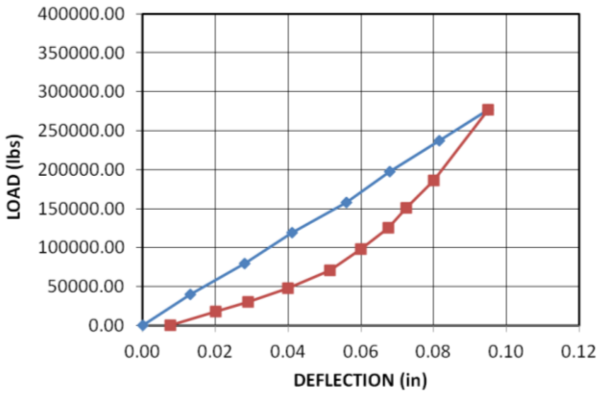

Fig.15. Testpile Graph for Testpile No.2, Test No.1

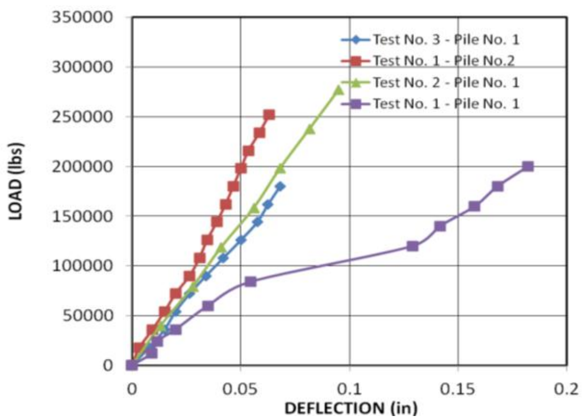

Fig.16. Loading Graphs for Piles No.1 and 2

PILE LOAD TEST RESULTS (July 14 and 15)

Two piles, Pile 3 and Pile 4, were tested on July 14. Testing of Pile 3, the first pile to be tested, was delayed for a few hours on account of inappropriate fittings between the compressor and the pump to the hydraulic jack. Further issues occurred during the testing of Pile 4 when the travel of the jack was exceeded resulting in loss of hydraulic fluid. These two events resulted in only two of the four (4) piles being tested on that day.

Only loading curves were obtained from these tests. When the load was increased from 244,000 lbs to 300,000 lbs on Pile 3, this pile could not sustain the increased load as the readings noted by the dial indicators began increasing rapidly and could not be readily read. The load of 244,000 lbs was therefore considered the maximum that the Pile 3 could sustain. It was also noted that this pile measured 18 inch in diameter at the top.

Pile 4 proceeded to accept loads to 300,000 lbs but this test had to be terminated after 315,000 lbs load as a result of leaking of fluid from the jack caused by a damaged seal. This resulted from the travel of the jack taken beyond a precautionary mark on the loading ram. The test was terminated at the 315,000 lbs load level. The unloading curve could not be taken. The final dial gauge reading for the loading curve was in close agreement to the movement of the pile as estimated from a laser level positioned to monitor the downward movement of the pile. The top of this pile was measured to be about 18 inches in diameter. Some lift-off of the reaction weights was noted when the pile load was increased to 315,000 lbs.

As a result of the damage to the jack, two replacement jacks were obtained for the testing of the remaining two piles (TP 5 and TP 6). The two jacks were positioned, one on each pile. This was a somewhat rainy day from the start of the working day. The load testing was not started until after the lunch interval as a result of repositioning of the reaction weights and drying out of the digital load indicator which became wet from the intermittent rainfall and was providing erratic readings without being connected to the jack.

A hair dryer was used to dry the electronic parts of the digital load indicator. An additional reaction slab was also added taking the reaction weights to 276 tons to counteract the lift-off problem. The load tests were not started until the digital load indicator could be reset to zero and the indicator tested and observed to be stable. The results of the load tests on these piles are shown in Figure 17. The tops of these piles were measured to be 22 inches in diameter in contrast to 18 inches measured for Piles 3 and 4.

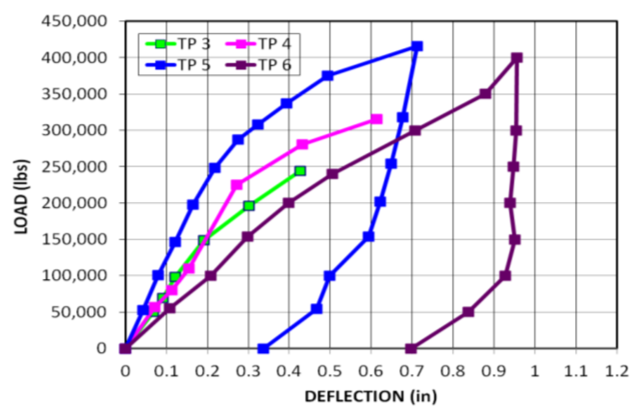

Fig.17. Pile Load test Results Piles 3 to 6

PILE LOAD TEST RESULTS (August 11)

For these tests the base plates between the jack and the H-sections were strengthened to avoid minor bending observed in the previous tests, additional strengthening was also undertaken between the H–section and the base plate by welding of stiffeners to the flange of the H-sections. The test results obtained from these test are shown in Fig. 18 below. These piles comprised the three-pile group under the waste heat flue. These piles were estimated to be of 9 ft embedded length.

These piles attained between 350,000 and 415,000 lbs after which the deflections began to increase toward one (1) inch, while Testpile 8 showed about half the movement for the a load of 416,000lbs.

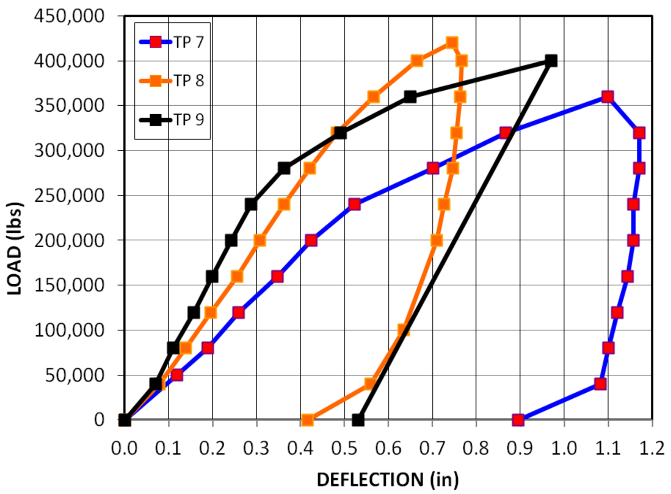

Fig.18. Pile Load Test Results – Piles 7 to 9

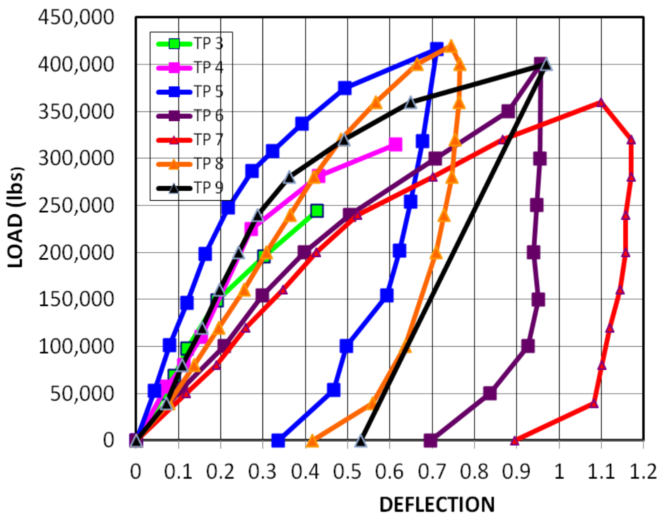

Fig 19: All Pile Load Tests Results of July and August

All pile load tests results (Fig.19) show general similarity in maximum loads attained by piles that were 22 inch in diameter. These loads ranged between 350,000 and 415,000 lbs. The curves showing the lower load results were piles that were 18 inch in diameter.

Despite that the maximum deflections for the 22 inch piles were different, all deflections at maximum loading ranged between 0.62 and 1.1 inch. For the 18 inch diameter piles the deflections ranged between 0.4 and 0.6 inch. Overall, the deflections of all the piles tested ranged for the most part between 0.4 and 1.1 inch.

The unloading curves of piles for which unloading was monitored, the non-recoverable deflections ranged between 0.3 and 0.9 inch with the majority between 0.3 and 0.6 inch. These values represent the movement of the pile, likely its toe, into the ground. Larger total deflections were obtained for the three-pile group with Test Pile 7 giving the largest deflection of 0.9 inches.

DISCUSSION OF PILE LOAD TEST RESULTS

Based on the deflections recorded from the piles under the pusher tracks, it was obvious that the movements under the applied loads resulted in elastic compression of the concrete and that there was none or negligible movement of the pile toe. This is not surprising since although the length of the piles were not accurately known, it was anticipated that the steel casing would be driven to terminate in the very dense till layer in the 1952 logs of boreholes 293 and 295 shown on the test pile location plan, Fig.1.

ELASTIC COMPRESSION OF PILES

The small movements recorded for the test piles and the small or no plastic deformation on unloading i.e., the pile rebounded to its original length when the load was removed (see intercept of unloading curve with deflection axis) for Test 2 and 3, Pile No. 26 and Test 1, Pile No. 62 indicated that the piles were only undergoing elastic compression of the concrete. As a check on the deflections recorded, the elastic deflection of the concrete was calculated using a pile diameter of 20 inches, pile length of 21 ft and the 28 day compressive strength of the concrete based on the design value as well as the strengths obtained from the core testing..

According to the notes on the design drawings, the piles shall have a minimum point diameter of 12 inch and minimum butt diameter of 14 in. There is no indication in the notes that the piles were of tapered construction although this pile type was popular at the time. The results are provided in tabular form in the Tables below for each of the two piles. The elastic compression was determined using the well known relationship below:

$$\text{Elastic Compression/Deflection} = PL/AE \qquad (1)$$

Where: P = Load, L = Pile length, A = Area of Pile, and E= Modulus of Elasticity of Concrete

The Modulus of Elasticity of the concrete was determined for the 2500 psi, 28-day compressive strength concrete by using the following relationship:

$$E = 4500\sqrt{f'_c} \qquad (2)$$

Where E = (Modulus of Elasticity) is in MPa. This was converted to psi by multiplying the result by 142.86. The relationship used is for normal concrete with a density of **2320 kg/m^3**.

For the compressive strengths obtained from the piles after 57 years (1952-2009), the Modulus of Elasticity was derived using the following relationship:

$$E=0.043y_c^{1.5}\sqrt{f'_c} \quad (3)$$

Where E is MPa, $\mathbf{y_c}$ is the density of concrete, and $\mathbf{f'_c}$ is the compressive strength of concrete. The density of concrete for Pile No. 1 was 2418 kg /m^3.

The defections obtained using the compressive strength of the cores was generally smaller than those with the design compressive strength and almost one half of the measured values from the pile load test. The smaller deflections obtained using the larger Modulus of Elasticity values are to be expected. The slightly larger deflection obtained when the maximum load was increased to 263,000 lbs for Test 2, pile No.1 is an indication that the test was proceeding to a failure zone. This is judged since there is a small residual strain when the loads were removed incrementally. Note in comparison Test No. 3 showed full elastic behaviour as there was no residual strain on off-loading. Hence, one can assume that less than around 180,000 lbs, the piles are expected to behave elastically.

Table.1. Deflections from Elastic Compression

Pile No	Test No	Max Load (lbs)	L (ft)	f'c (psi)	E (psi)	D (in)
1	1	200000	19	2500	2,689,264	0.044 (.018)
1	2	263000	19	2500	2,689,264	0.059 (.094)
	3	180000	19	2500	2,689,264	0.040 (.068)
2	1	250000	19	2500	2,689,264	0.056 (.058)
1	1	200000	19	5678	4,601,457	0.026 (.18)
2	2	263000	19	5678	4,601,457	0.036 (.094)
3	3	180000	19	5678	4,601,457	0.031 (.068)
2	1	250000	19	9227	6,192,101	0.032 (.024)

ULTIMATE GEOTECHNICAL RESISTANCE OF PILES 1 AND 2

On the observation that the movements of the tested piles under load were relatively small, the typical curve fitting for determining the ultimate resistance of the piles was not undertaken as these are generally more applicable to piles that a have undergone some appreciable toe movement. For example, the popularly used Davisson's offset method would require an offset line at 0.15" plus a value equal to diameter of the pile divided by 120. Hence, this would result in a deflection 0.32 in (0.15"+ 0.17"). This deflection exceeds the deflections obtained from the tests.

Of the two other popular methods, the Brinch Hansen - Failure criterion and Chin Failure criterion, the Brinch-Hansen method suggests that the maximum load applied would be the geotechnical resistance of the piles since no plunging failure of the piles occurred. The Chin method is seldom used since the values are often higher than the maximum load applied.

The Brinch Hansen approach was used to determine the failure load. The maximum applied load of 263,000 lbs for Pile 1 and a load of 250,000 lbs are considered to represent the ultimate geotechnical resistance at ULS for these piles. Using an average value of 256,500 lbs the factored geotechnical resistance would be around 150,000 lbs (67 tons) at ULS using a geotechnical resistance factor of 0.6.

In terms of Working Stress Design (WSD), the allowable geotechnical resistance would be 125,500 lbs (56 tons) using a Factor of Safety of 2.0. For a Factor of Safety of 1.5 which can be used as well, the allowable geotechnical resistance would be 167,000 lbs (75 tons).

As noted since the deflections are small, a much higher geotechnical resistance can be used if the design factored loads require a higher factored geotechnical resistance. However, if this is the case, then this should be discussed to determine a suitable increased factored geotechnical

The allowable loads during the period of construction of the No. 6 Coke Battery would have been based on a dynamic driving formula. The Engineering News Record Formula was popularly used. Based on the criterion for driving of 6 to 7 blows per inch provided on the design drawings, the allowable load was determined for the Vulcan No. 1 Hammer for the prescribed driving criteria to be 58 tons and 64 tons for 6 and 7 blows, respectively. These values are in excess of the design load of 45 tons.

These values are based on the driving of the outer steel shell to toe elevation i.e., into the hard till. Generally, the Factor of Safety that is used with the Engineering News Formula is around 6. However, there are issues with this Dynamic Formula which may under predict or over predict the pile capacity. A lot depends on the efficiency of the hammer since an inefficient working hammer can show that the criterion is achieved with less energy applied and hence an incorrect safe load. This capacity does not account for the resistance of the concrete which becomes important if the pile base is unyielding and hence the pile can be treated as a structural member. Assuming that the piles were driven to the same set and with the prescribed energy then one can assume that the individual piles are capable of providing the proposed design load.

ULTIMATE GEOTECHNICAL RESISTANCE OF PILES 3 TO 9

The maximum loads on the pile-movement graphs were taken to represent the failure loads of the piles since it was generally observed that for piles taken to 400,000 lbs and larger the dial indicators showed substantial increased movements. For the last three piles tested in August 11, the movements of these piles recorded by the dial indicators were visually checked against movement of the pile using a laser beam. The change in the position of the laser beam location between the beginning and end of a test was found to be generally of the same order and hence the load curves obtained are felt to be realistic and curves reliable to be used for design purposes. The geotechnical resistances, which represent the ultimate resistance of the piles, are summarized in the Table 1. A factor of 2240 lbs per ton was used in converting pounds to tons.

FACTORED GEOTECHNICAL RESISTANCE OF PILES

Using a geotechnical resistance factor of 0.6 recommended by the Canadian Foundation Engineering Manual (2006), the factored resistances for the piles are provided in Table 2. It should be noted that there are some prevailing opinions that a geotechnical resistance factor of 0.8 should be applied to the results of load tests. Use of this factor for these piles seems to be realistic based on the load-settlement relationships obtained from the test results including the 18 inch diameter piles. However, the use of a higher resistance factor than the Code recommendation at this time requires that a reliability based design (RBD) evaluation be undertaken. This evaluation is not within the scope of this report.

As noted, except for the two smaller diameter piles tested in July 14 and 15, the factored geotechnical resistances of the larger diameter piles are generally slightly larger than 90 tons. As noted previously, load testing of Pile No. 2 had abandoned as a result of jack problems. Conceivably, this pile could have achieved the desired 90 tons. The only pile falling short of this target was Pile No.1, but the result is not unusual.

Since both the short piles (9 ft) and the long piles (12 ft) provide similar resistances, it is the opinion that the major geotechnical resistance of these piles are obtained from toe resistance being embedded in very dense till as noted from the two borehole logs in Drawing 2 which are reasonably consistent with the subsurface stratigraphy shown by other boreholes done within the No. 6 Coke Battery.

One component of the design using the ULS approach is to ensure that the factored structural loads do not exceed the factored geotechnical resistance. Another component is the requirement of serviceability limit state whereby settlement and differential settlements are components that are required to be checked to ensure that these are within acceptable limits. These requirements would normally be set by the structural engineer.

Table.2. Factored Geotechnical Resistance

Pile No	**Ultimate Geotechnical Resistance (lbs)(tons)**	**Factored Geotechnical Resistance (lbs) (tons)**	**Remarks Dia (length)**
3	244,000 (109)	122,000 (65)	18”dia (12ft)
4	315, 000 (141)	157,500 (85)	18”dia (12 ft)
5	416,000 (186)	208,000 (111)	22”dia (12 ft)
6	400,000 (178)	200,000 (107)	22”dia (12 ft)
7	360,000 (161)	180,000 (97)	22”dia (9 ft)
8	420,000 (188)	210,000 (113)	22”dia (9 ft)
9	400,000 (186)	200,000 (111)	22”dia 9 ft)

One component of the design using the ULS approach is to ensure that the factored structural loads do not exceed the factored geotechnical resistance. Another component is the requirement of serviceability limit state whereby settlement and differential settlements are components that are required to be checked to ensure that these are within acceptable limits. These requirements would normally be set by the structural engineer.

SERVICEABILITY LIMIT STATE

Since the service loads generally utilize a load factor of unity, then it is expected that the Service loads will be smaller than the factored loads of the ULS. With lower loads than the factored geotechnical resistance the settlement of the piles are expected not to exceed 0.45 inches if we look at the desirable maximum load requirement of 90 tons per pile and smaller loads in some cases depending on the loading of the infrastructure to be proposed. For a proper determination of the magnitude of total and differential settlement, the service loads will be required. Overall, the load curves shown in Fig.19 can be used by the structural designer to assess the settlement and differential settlement characteristics of the ground under the proposed loads.

WORKING STRESS DESIGN

The typical working stress design recommendations for determining the allowable geotechnical resistance was determined by applying a Factor of Safety (FOS) to the ultimate geotechnical resistance. Common (FOS) would vary from 1.5 to 2 with 2 being a more common value. The allowable geotechnical resistance would be determined by dividing the ultimate geotechnical resistance by the FOS. Table 3 shows the allowable geotechnical resistance of the piles for a FOS of 2. In determining settlement under these loads, this can be obtained from the load test results. Again, it is not expected that the values would be outside the range discussed for the Serviceability Limit States.

Table. 3. Allowable Geotechnical Resistance

Pile No	Ultimate Geotechnical Resistance (lbs)(tons)	Allowable Geotechnical Resistance (lbs) (tons)	Remarks Dia (length)
3	244,000 (109)	122,000 (61)	18"dia (12ft)
4	315, 000 (141)	157.500 (71)	18"dia (12 ft)
5	416,000 (186)	208,000 (93)	22"dia (12 ft)
6	400,000 (178)	200,000 (93)	22"dia (12 ft)
7	360,000 (161)	180,000 (81)	22"dia (9 ft)
8	420,000 (188)	210,000 (94)	22"dia (9 ft)
9	400,000 (186)	200,000 (93)	22"dia 9 ft)

STRUCTURAL RESISTANCE AND FACTORED STRUCTURAL RESISTANCE OF TESTED PILES

If we consider that the piles were terminated in material that is non-yielding, then we can examine the structural resistance of the composite pile and apply a structural resistance factor. In the absence of the characteristics of the outer steel shell, the structural resistance of the pile can be determined from the concrete strength as follows:

Factored Structural Resistance = 0.85 x 0.6 x 380 x 2500 = 484,500 lbs (216 tons) for 22 inch diameter piles and a resistance of 0.85 x 0.6 x 254 x 2500 = 323,850 lbs (145 tons) for 18 inch diameter piles.

Much higher values would be obtained if we utilize the core compressive strengths of the derived from testing the concrete cores. This would lead to approximately 2.5 to 3 times the values above.

The structural resistance values are higher than the factored geotechnical resistances and hence the factored geotechnical resistances would govern in the design for the ULS case.

Despite the lack of information on the actual pile sizes throughout the existing coke battery Foundation, the disposition of the pile load test curves have shown that the existing piles were installed with care otherwise one would have expected to see more variability in the test results. In any production piling, only a few pile load tests are often undertaken. The number of tests done and the length of piles tested would appear to cover the spectrum of piles within the #6 Coke Battery Foundations. From review of the construction drawings, the three-pile group under the waste heat flue appears to be the only ones that are 9 ft long. The majority of piles were likely of 12 ft embedded length.

The tests undertaken have considered axial compressive resistances of the piles only. No lateral load or uplift considerations were examined as it was generally understood that the infrastructure would not be subjected to such loads. These requirements have to be evaluated by the structural designer. If the piles are required to take non-concentric loads or uplift loads then geotechnical resistances of the existing piles may need to be re-evaluated by static analysis approach.

Overall, based on engineering judgment, one can assume from the pile load testing results that the existing foundation piles constructed in or around 1952 piles were competently undertaken and can sustain axial loads greater that what they were likely designed for at the time to accommodate.

SUMMARY AND CONCLUSIONS

The desire to reconstruct the previously demolished No. 6 Coke Battery to a level higher than that of the previous structure required the assessment and evaluation of the pile foundations supporting the existing structure. Static axial pile load tests were considered to be the best approach in determining the geotechnical resistance of the existing piles. According to the 1952 design drawings the foundation support consisted of steel cased concrete filled piles. These piles were to be driven to a resistance of 6 to 7 blow per inch for the last three (3) inches of penetration with a No.1 Vulcan Hammer or equal.

In reviewing this historic pile type it was determined that this cast in place pile was a step taper, thin shell corrugated wall ringed pile as installed by Raymond International. This pile is constructed by driving the outside shell into a pre-bored hole by internal methods. The use of a steel mandrel inserted into the shell permits hard driving and the driving energy is transmitted directly to the tip. Concreting is undertaken after the pile is driven. Since these piles do not require longitudinal steel reinforcing when the full length is enclosed by soil strata Their use is limited when lateral support is lacking.

In order to undertake the static axial pile load tests considerable time and expenditure were involved in locating the piles whose tops were buried, in some cases, in excess of 12 feet below existing below the existing ground level and below massive reinforced concrete infrastructure of slabs and pile caps. Site dewatering and concrete breakages to expose the piles were the most time consuming aspects of the foundation demolition.

Despite the lack of information on the actual pile sizes constructed throughout the existing Coke Battery Foundation, the pile load test curves have shown that the existing piles were installed with care otherwise one would have expected to see more variability in the test results. In any production piling, only a few pile load tests are often undertaken.

The number of tests done and the length of piles tested would appear to cover the spectrum of piles within the #6 Coke Battery Foundations. From review of the construction drawings the three-pile group under the waste heat flue appears to be the only ones that are 9 ft long. The majority of piles would likely have been embedded 12 ft.

The tests undertaken have considered axial compressive resistances of the piles only. No lateral load or uplift considerations were examined as it was generally understood that the infrastructure would not be subjected to such loads. However, these requirements have to be evaluated by the structural designer. If the piles are required to take eccentric loads or uplift loads then geotechnical resistances of the existing piles may need to be re-evaluated by static analysis approach.

Overall, based on engineering judgment, one can assume from the pile load testing results that the existing foundation piles constructed in or around 1952 piles were competently undertaken and can sustain axial loads greater that what they were likely designed for at the time to accommodate.

This case study demonstrates the difficulties that could arise in testing and evaluating historic foundations and the need to have some forward thinking in the design and construction of new infrastructure so that substructure elements can be readily tested and evaluated in time.

The re-use of foundations should be considered both when installing new foundations or re-using existing foundations. To future-proof these foundations they need to be documented and an understanding of their behaviour gathered. A documentation system needs to be developed to record all the necessary data to enable a foundation to be re-used and new 'smart' instrumentation that can monitor a building's behaviour during its life thereby demonstrating the foundation behaviour and its potential for re-use. Much of this type of work has been initiated in 2003 by the European Community under project "RuFUS" (Re-use of Foundations for Urban Sites).

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