

L3**L4**

Depth (m)	Soil Conditions	Depth (m)	Soil Conditions
0-3	Stiff Silt	0-5.5	Silt and Sandy Silt
3-7.3	Dense Gravelly Sand	5.5-9.8	Med-Dense Gravel
7.3-11.5	Very Dense Gravel	9.8-14.6	Loose-Med Sand
11.5-16	Medium Sand	14.6-22.8	Dense Sandy Gravel

H3**H4**

Depth (m)	Soil Conditions	Depth (m)	Soil Conditions
0-3.1	Fine Sand	0-5.8	Soft Silt
3.1-4.9	Stiff Silt	5.8-11.3	Very Dense Gravel
4.9-10.6	Gravelly Sand	11.3-15.9	Fine Sands
10.6-14.3	V.Dense Gravel		

Table 1 Summary of Soil Conditions at the Load Tested Shafts

Offshore Pressuremeter Testing

A TEXAM hydraulic control unit was used operating the long NX probe of 1850 cc deflated volume. The use of the PMT was directed toward the upper 18m of all deposits, which controls lateral response of the shafts. Drilling with NJ rod and the circulation of drilling mud was protected from the 5.5m/s river currents inside a 203mm diameter conductor pipe set below the river bottom. From a total of 18 prepared test sections 5 collapsed under artesian pressure in the clean sands and some occasional squeezing of the hole was evident. Consistent with all insitu testing in coarse gravels the PMT does have limitations in achieving the maximum success rate of highest quality data in gravelly soil. Table 2 illustrates the range of PMT properties; net limit pressure P_L^* , initial modulus, E_o , and reload modulus, E_R , for the offshore boreholes in all soil encountered. By following procedure B of ASTM D4719-87 no creep pressures are measured. The overall success for the tests, conducted over 22m of fast moving water, in these difficult conditions and aggressive soils was remarkable, and provided quality soil characterization for P-y work.

With the exception of the near surface silts all soils were revealed as showing fully drained behavior. As a way of assessing test quality in these material a maximum 3 star (***) rating is used, and shown in Table 2.

Test No.	Depth Below Mudline (m)	Initial Modulus E ₀ (kPa)	Net Limit Pres. P _L (kPa)	Ratio E ₀ /P _L	Reload Modulus E _r (kPa)	Soil Type
3 ***	4.6	5506	512	10.7	23078	Silt
6 ***	3.2	5506	613	9.0	28250	Silt
7 ***	5.5	6560	670	9.8	20396	Silt
9 **	7.4	>4980	991	5.0	12310	Sandy Silt
10no star	9.75	>10500	----	----	-----	Silty Gravel
11 **	7.0	9528	1110	8.6	20636	Gravelly Sand
12 **	8.23	16758	2050	8.2	72970	Sandy Gravel
15 ***	8.96	9145	1039	8.8	53201	Sandy Gravel
16no star	11.28	>1700	----	----	>7600	Loose Gravel
19 ***	16.03	4596	411	11.2	16087	Silt
20 ***	11.43	8427	527	16	25376	Sandy Silt
22 **	16.67	16280	1603	10.1	33611	Gravel
25 ***	21.31	5315	723	7.4	34760	Sand

NOTE: Test numbers not listed are either calibration tests, or tests which were not be attempted due to heave

Table 2: Summary of Offshore Pressuremeter Tests

The full *** rating is given to the highest quality PMT test curve which contains a well defined modulus, sufficient plastic behaviour to approach a limit pressure, P_L , and shows no adverse effects from borehole preparation.

DESIGN LOADING AND P-Y METHODOLOGY

Predicting the resistance to lateral loading for a shaft is a complex, non-linear, soil-structure interaction problem, which has received considerable attention in the last 20 years. Significant parameters include the magnitude, duration and frequency of the loads, shaft stiffness and geometry, and the soil layering and properties. Since the low flume (L) is submerged during projected frequent high river elevations and the high flume (H) by flood conditions, repeat cycle downstream

loads, of low frequency, are expected. In addition, load from river currents to the submerged portion of the shafts above the riverbed and combined loads resulting from ice, floating debris, wind and seismic must all be considered. Tolerable flume joint lateral movements must be restrained to 0.76m differential to the river shore. The foundation type was restricted to the single large shafts to minimize adverse hydraulic effects, such as eddying, and avoid providing predator fish habitat that threaten the juveniles. Due to these considerations, the non-linear LPILE code was selected using PMT based P-y procedures.

P-Y Procedures

The method chosen as most suitable to capture these large shafts was the Briaud/Smith method (Smith 1987, Briaud 1992). These procedures take account of installation disturbance, soil layering, pile stiffness, and the depth of reduced soil resistance, D_c , effects. The method calls for construction of both the 'front' pressure (Q-y) and side shear (F-y) mobilization curves, which are added for the P-y curve, after slope and disturbance effects are included, Figure 2.

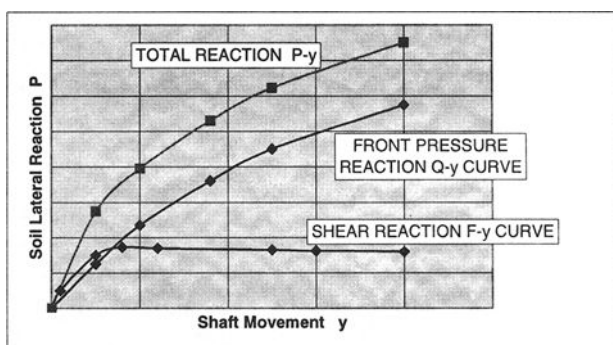


Figure 2 Front Pressure and Side Shear Components of the P-y Curve

Use of the SPT together with density descriptions was made as a correlation tool to complete a full set of PMT curves for each soil at each location. Tolerable movements at the mudline would be restricted to less than 4% of the shaft diameter. The depth of reduced resistance, D_c , varied between 1 and 1.8 pile diameters in depth, and controls the important near surface curves. The Briaud/Smith procedure calls for use of the initial PMT modulus on zero displacement piles to capture the lack of any driving benefits. Most empirical procedures are derived from load tests in sand on *closed end* full displacement piles. Closed end piles would feel the beneficial increase in horizontal stress and modulus from higher confining pressures and soil densification. These conditions would not apply at the Bonneville tests due to the vibrodriven installation of the open permanent casing and possible liquefaction

of clean granular deposits, including gravels. The choice of the appropriate early strain soil modulus, E_s , is critical to predict movement of these shafts. Most theoretical work supports the conclusion that initial E_s values are *not* functions of shaft diameter.

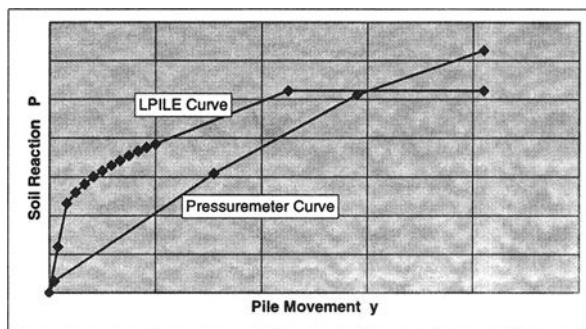


Figure 3 P-y Curves at 1.5 m Depth for H3 Shaft

A typical P-y curve at a depth of 1.5m (1/3 way down the Dc effect) at the H3 shaft is shown in Figure 3 which compares the PMT curve and LPILE subroutine generated curve for medium sand with $\phi = 36$ degrees and $k = 16.3 \text{ MN/m}^3$. The SPT blow count at this depth was 24. Using the PMT based curves prediction of shaft riverbed, and flume, deflections were made for all offshore shafts by the structural engineers with LPILE. These deflections were close to the maximum tolerated and raised concerns about the flume movement and the accumulated effects of repeat cycles over the structure's 50-year life. The geotechnical design team assembled a database of 58 lateral load pile tests in the technical literature, of which 24 were in granular soils and contained cyclic loading. A preliminary relationship was established between the initial first cycle deflection, rebound, and repeat load deflection increase for movements less than 5% of the shaft diameter.

The decision to proceed with a full-scale proof load test on the production shafts was based on the need to provide the following:

- Confirmation that first cycle deflections were correctly predicted by LPILE, and that the diameter effect are properly represented with PMT P-Y procedure.
- An understanding of pile behaviour under repeat, low frequency, lateral cyclic loads and high moments.
- Evaluation of possible creep under constant load.
- Provide a basis to activate, or delete, the large proposed 'Drag Collar' improvement option contained in the contract.

LOAD TESTING PROCEDURES AND INSTRUMENTATION

Test Setup

Of the six shafts vibro-driven in the river, four had significant 'stick up' heights and predicted deflections which were of concern. These were H3, H4, L3 and L4. Testing could be achieved by passing a multi-strand steel cable through steel sleeves for the shaft pairs H3 to H4, and L3 to L4, which could then be jacked toward each other for reaction, thus placing the cable in tension, Figure 4.

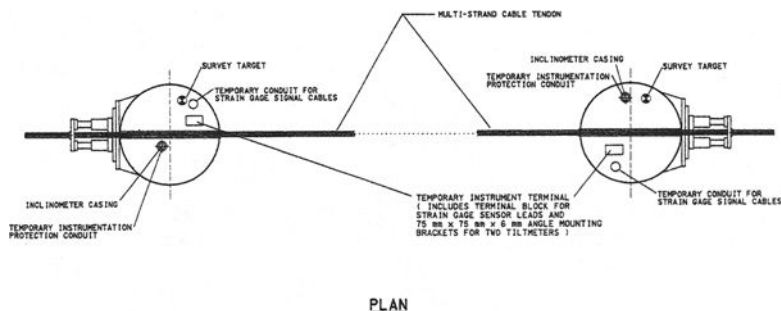


Figure 4: Plan of Load Testing Configuration in Tension

The load would be applied perpendicular to the river current and in line with the proposed flume. This prevented any shaft preset and soil stress in the future downstream current loading direction. Thus, a total of 4 load tests would be conducted on offshore shafts of diameters much larger than currently reported in the literature.

The shaft placement and load-testing requirement called for the contractor to install a temporary support trestle clear of the fluctuating river elevations. This 105 m x 22 m trestle was timber decked and provided access to all construction plant. Eighty 0.91m diameter steel pipe piles supported the trestle. The 3.05 m production shafts were concrete filled up to the final design elevation with additional stiffening into the extended casing portion used for load application via the cable. An anchor slab was constructed around the 200mm diameter steel sleeve and stiffened the shaft where the rams reacted externally against the shaft.

A 40m long, 19 strand, multi-cable tendon with 4.0 MN capacity was hung to twin 1.8MN jacks at the shaft, giving a 3.6 MN load capacity to each shaft. The anchor head assembly comprised bearing plates, hydraulic jacks, yoke, load cell and anchor head. Each jack had a maximum throw of 0.61m, which provided for a total cable extension of 1.22m. At the peak anticipated load the cable stretch in tension would consume 260 mm leaving a total combined shaft deflection, at the point of load application, of 0.96m. An LPILE prediction for this loading geometry

configuration indicated this amount of deflection at the cable point would generate of the order 40mm to 70mm shaft deflections at the riverbed.

Instrumentation

A wide array of instrumentation was present on each of the 4 shafts. These comprised tiltmeters, strain gages, inclinometers, survey points and load cells. Geokon model 6350 vibrating wire tiltmeters were installed at the shaft top, in pairs, to measure tilt in the loading direction and the orthogonal direction. Thirty-six Geokon vibrating wire strain gages were installed on H4 and L4, and thirty-two on H3 and L3. These were placed in sets of 4 at the quadrant points around the rebar cage and protected during installation. Inclinometers were used to monitor shaft deflected shape after each load, and unload, application. On H3 and L3 a SINCO 70mm casing was installed and on H4 and L4 a 85mm casing was installed. These were grouted in place at the base into NX sized core holes drilled through the concrete and extended into bedrock. (Two survey points on each shaft provided additional data and a check to the inclinometers on the top of shaft.) Finally, a vibrating wire load cell was placed in line with the anchor head and yoke at the North end of each cable tendon, at H3 and L3.

An 80 channel Geomation 2380 automated data acquisition/reduction system monitored strain gages, tilt meters and load cell every 2 minutes. Four 24-pair direct burial type signal cables were used to connect instrumentation to the Geomation 2380. Frequency of monitoring the survey points was 10 to 15 minutes. The Contractor's personnel operated the hydraulic load system for applying jack pressure.

Test Procedure

A number of limiting 'red flag' (stop test) criteria were agreed upon by the team. The maximum allowable peak mudline deflection of 100mm, or, a maximum allowable permanent deflection of 50mm, controlled the allowable load level. Test loads governed by the jacks could not exceed 3.6 MN. By way of comparison peak equivalent 100-year flood loads were estimated at 3.1 MN for shaft H4 and 2.67 MN at L4.

Each test load was initiated at 222 kN and increased in increments of 444 kN. A minimum unload condition of 222 kN was maintained for the entire test to ensure no slippage in the stressing system. The typical data acquisition sequence followed:

- Load application, shaft deformation electronically tracked till stabilization reached ($\frac{1}{2}$ to 1 hour).
- Hydraulic load and survey data gathered
- Inclinometer readings on both piers, (1 hour)
- Inclinometer data downloaded from SINCO Datamate and processed by Digipro®

- Tiltmeter, load cell and strain gage data extracted
- A second set of hydraulic loads & survey data entered to establish any shaft 'drift'

To assist field crews in control of the test and allow data evaluation by the design team a 'decision tree' was employed to establish the proper course of action after each load step. The full series of tests conducted on the H shafts lasted 9 days and, following 7 days of demobilization and transfer, a further 9 days to the L shafts. Typically, the loading sequence varied between 2 hours to 6 hours with a creep portion of 24 hours built into the L tests at about the 1.78 MN load.

Table 3 presents the unrestrained 'stick up' height from riverbed to the loading cable, load levels and number of cycles for all shafts.

SHAFT: STICK-UP HEIGHT (m) H3:23.8, H4:24.5, L3:24.6, L4:24.2

Peak Load (MN)	.445	.890	1.34	1.78	2.22
H3 & H4	2	4	5	3	2
# of cycles					
L3 & L4	6	6	5	1*	2
# of cycles					

*24 hour creep test.

Table 3 Unload-Reload Test Summary

. Each unload cycle was taken down to the baseline minimum of 222kN prior to reloading up to the next selected load level.

LOAD TEST RESULTS AND DESIGN SERVICE LIFE

Results

Figures 5 and 6 show the mudline deflection, measured by the inclinometer against applied load, recorded by the load cell, for L and H shafts respectively. The plots confirm nonlinearity is present at all these comparatively low load levels for such large shafts. This confirms that nonlinear P-y based LPILE application is appropriate even with peak-measured deflections less than 3½% of shaft diameter. The LPILE reported predictions are based on subroutines in the software and follow the procedures given by Reese et. al. (1974) for submerged granular soils. It does appear the vibro installation procedure for these shafts, with zero displacement, leaves these sands and gravels in a loose state following possible partial liquefaction. This seriously damages the small strain modulus and produces *higher* deflections.

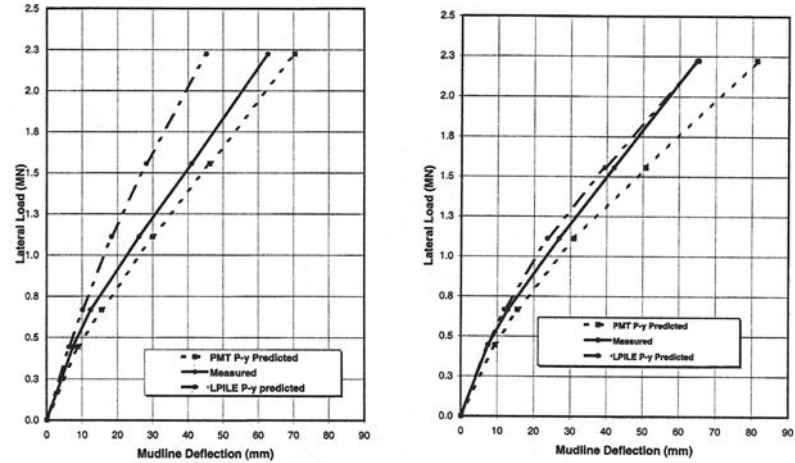


Figure 5: Measured and Predicted Load Deflection for L3 and L4

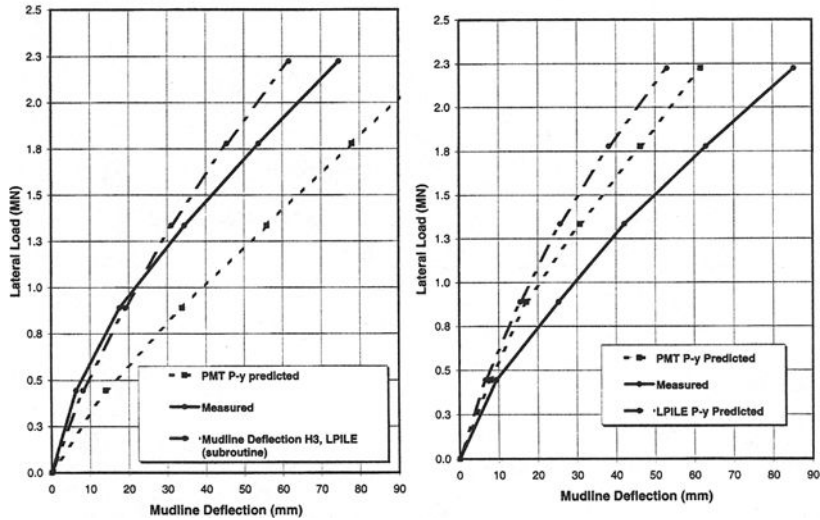


Figure 6: Measured and Predicted Load Deflection for H3 and H4

Figures 5 and 6 also show the predicted response from both P-y methods. The PMT procedures for P-y construction are shown to have done very well in predicting the response, with acceptable small conservatism. Taken together the LPILE subroutines and PMT based methods have quite well 'bracketed' the shafts response in 3 of the 4 load tests. It should be noted that at the time of shaft

installation H4 encountered a large boulder, which required rock chisels to break up and facilitate removal from inside the open casing. This is believed to have further loosened the gravels and produced higher deflections under load. All other shafts were vibro-driven continuously without interruption.

Design Service Life

To model the effects to peak deflection from repeat load flood cycles the popular power law relationship below was used:

$$y(N) = y(1) \times N^{\alpha} \dots\dots\dots \text{Eq. (1)}$$

Using historic hydrologic data from Bonneville Dam, a statistical model was developed by the Portland District to represent the expected variations of river elevation from flood and electric power generation requirements. These variations were converted by the design team to shaft load levels and the number of load cycles projected over a 50-year design life. These are presented in Table 4 for both H and L shafts. The higher number of loads and cycles at the L shafts are caused by the frequent submerged conditions for the low elevation flume.

L3 and L4		H3 and H4	
Design Load (MN)	No. of Cycles	Design Load MN	No. of Cycles
0.756	2660	0.267	40
1.023	362	0.556	49
1.29	72	0.734	29
1.82	144	1.0	35
2.36	26	1.334	42
2.713	2	1.779	4

Table 4: High (H) and (L) Shaft Loads and Cycles for 50-year life

Back calculated α values for each of the 4 shafts load tested, and subjected to the number of cycles presented earlier, ranged from 0.07 to 0.1. From the site-specific α values, and equation 1, the mudline and flume deflection could be calculated for the load ranges given in Table 4. Predicted deflection at L3, L4, H3 and H4 were 45%, 64%, 30%, and 82% respectively of the maximum allowable using the actual load test result (not LPILE predictions), and equation 1. Thus total deflections for all shafts over 50 years were less than the maximum allowable, which if exceeded would have rendered the flume inoperable.

The effect of the boulder and subsequent disturbance at H4 is seen in the highest percentage of allowable deflection consumed by the expected load cycles over 50 years. All shafts have acceptable factors of safety. As a result of the 24-hour constant load test at 1.78 MN for L3 & L4, creep was not judged to be of concern. At the present, the instrumentation continues to be monitored to track the

actual performance of the shafts.

SUMMARY

From a review of the literature the Authors believe these load tests set a new record in diameter for load testing under horizontal load. As a result of the site investigation and full-scale offshore cyclic load testing to these 3.05m diameter shafts a number of conclusions can be advanced.

- These large diameter, zero displacement shafts, installed by vibro techniques lower the pile lateral soil modulus. Great care should be exercised in constructing P-y curves, and some judgement applied to selecting the initial modulus of subgrade reaction. Backcalculated constant of subgrade reaction values at these shafts are around 2.3 MN/m^3 for the 2.22 MN load.
- Successful high quality pre-bored pressuremeter tests can be made offshore in silt/sand/gravel deposits.
- Pressuremeter based P-y procedures were successfully employed and satisfactorily accommodated the pile diameter effect.
- Calibration of predictive models by full scale testing permitted design life deflection issues to be resolved. The cost saving to the Corps of Engineers from deleting the Drag Collar retrofit option amounted to \$1M.
- The need for effective teamwork between owner-structural and geotechnical consultants is of paramount importance, particularly on soil/structure issues. The Portland District Corps of Engineers technical staff commented on the excellent teamwork demonstrated in this project.
- Consideration should be given to ensuring resources are available to monitor large foundation throughout the design service life. Thus future data analyses can confirm the long term performance is on target to ensure a full service life, or if flood load produce excessive deflection remedial measures can be planned well in advance.

APPENDIX I : REFERENCES

Briaud, J.L. (1992) *The Pressuremeter*, Balkema Books, pp 144-151

Reese, L., Cox, W. R. and Koop, F.D. (1974) "Analysis of Laterally Loaded Piles in Sand", OTC Paper 2312, Proc., 7th Offshore Technology Conference, Houston, TX

Smith, T.D. (1987) "Friction Mobilization F-y Curves for Laterally Loaded Piles from the Pressuremeter", Proc., Prediction and Performance in Geotechnical Engineering, Calgary, Canada. pp 89-96