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# Lateral Load Prediction and Testing of 3.05m Dia. Shafts

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## ABSTRACT

Four 3.05m diameter permanent steel-cased shafts were laterally load-tested over water in the Columbia River, downstream of Bonneville Dam, Oregon. The site investigation into the subsurface gravels, sandy gravels and near mudline silts included pre-bored pressuremeter tests. Design life lateral loads arrive from the pile supported concrete fish flume superstructure being submerged during periods of high river flows, producing large repeat loads to the shafts. The industry standard LPILE code was employed with *user defined* P-y curves constructed from Pressuremeter data to predict the first cycle load deflection response. Predicted mudline deflection & slopes were close to the maximum structurally tolerated by the flume, 25m above the mudline. Full-scale lateral instrumented load testing on the production shafts validated the analysis and P-y curves up to the maximum 2.22MN. A power law model was used to evaluate the repeat load cycles in the load test and establish the long term accumulated cyclic deflection. Optional foundation improvements were also examined, including a 12.2m diameter concrete Drag Collar designed from the Pressuremeter data.

## INTRODUCTION

Bonneville Dam is a large hydroelectric scheme generating 558 MW of power and situated 55 km. east of Portland, Oregon, on the Columbia River. To improve the survival rate of juvenile fish hatching upstream a $38M Juvenile Fish Bypass System (JBS) is nearing completion to carry fish around the 2 powerhouses, 2 navigation locks and the spillways. The JBS safely transports fish 2km downstream

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to be discharged from a concrete flume above swift water currents. To maintain minimum elevations above the waterline the square concrete flume has twin Outfalls over water; one for normal water levels (L) and a second for high river, flood, conditions (H). These flood stages, based on spring snow and variable power generation demands fully submerge the low flume seasonally below water elevations and provide high hydraulic forces. These forces can be classified as repeat, one way, cyclic monotonic loads, always in the downstream direction and normal to the flume axis. The high flume may also see cyclic loads with 100year frequency floods, but much less often. Flume invert elevations above the river mudline are approximately 25m to the high (H) flume and 20m to the low (L) flume. To support each of the 125m elevated flume lengths over water 3 large shafts of 3.05m diameter were chosen, carrying flume spans of 35m on each H & L flume, Figure 1.

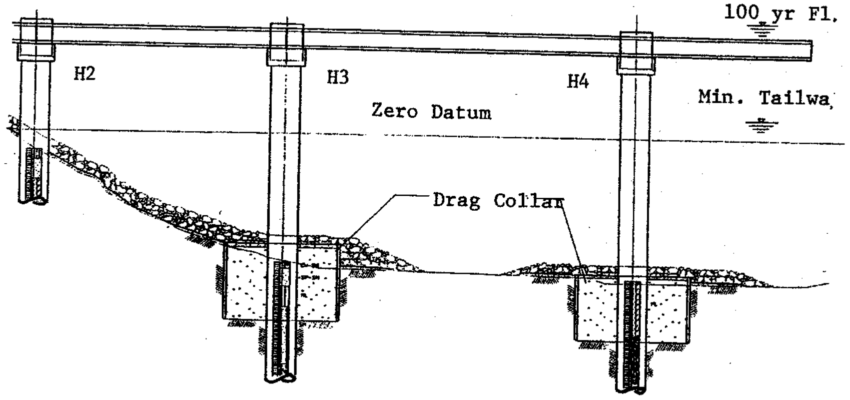



Figure1: Elevation of the High Outfall Flume at Scale 1:250

Generalized subsurface conditions comprise alternating layers of coarse grained medium to dense gravels, sandy gravels, and some sand, below a near surface soft silt zone up to 6m in thickness. This silt zone at shafts H4 and L4 has a depth of twice the pile diameter and dominates the lateral load performance. With the predominance of gravels and sandy gravels a permanent steel casing 25mm thick was vibro driven and then chiseled into bedrock at depth between 27m and up to 35 m below river bottom. After removing the soil plug a rebar cage was lowered and concrete tremied up to the flume elevation.

Tolerable structural flume deflections are of the order 0.75 m. With the 20m to 25m stickup the mudline slope, as well as mudline deflection, contribute to flume deflections. Limited in-water work periods, predator fish, environmental concerns and hydraulic considerations prevented most accepted design approaches to stiffen the deflection performance of the shafts. Predicted shaft and resulting flume deflections necessitated consideration of a variety of construction alternatives. The selected option for construction bids, was a 12.2 m diameter mass concrete 'Drag Collar' set into the surface silts and mobilizing frictional drag by bearing on the gravels. The purpose of the four full-scale instrumented load tests was 3 fold:

1) Validate the PMT based LPILE predictions for first cycle loadings.

2) To gain unload-reload cyclic load and creep data for evaluation of accumulated deflection over a 50 years service life for the flumes.

3) To determine if the concrete Drag Collars were required to be installed by the contractor at an additional $1M project cost.

The load testing took place during an approved 'in-water' work period from a purpose built driven pipe pile supported trestle in May 1998[4].

## OFFSHORE INVESTIGATION AND SHAFT CONSTRUCTION

### Generalized Conditions

The subsurface investigation was conducted in 2 phases. Phase 1 comprised two onshore exploratory boreholes using an open ended Becker hammer drill at the location of H1 and L1 shafts. One of these boreholes also included SPT and pre-bored pressuremeter testing (PMT) by mud rotary techniques through the Becker casing. These tests confirmed the feasibility of PMT for all the offshore shafts at H2 to H4 and L2 to L4 as the prime investigative tool to characterize the gravels, sandy gravels and silts. The Phase 2 offshore investigation was undertaken from a spud located barge, with further anchorage in swift current provided by a Columbia river tug during exceptionally heavy river flows in December 1996. The testing program included 20 PMT tests with occasional SPT tests below the PMT test zone, when practical, in the silts. Some limited CPT testing in artesian pressure bearing sands from 10m to 16m, encountered at H3, and diamond coring in the dense gravels above the sedimentary bedrock were also made.

Conditions along both the H and L flume alignments were similar with silts, interbedded sands, gravelly sand and sandy gravel. Each shaft location had a dedicated borehole in which PMTs and SPTs were taken. Table 1 indicates the variety of soil conditions offshore. It can be seen most shafts would encounter near surface silts and loose to medium sands during installation. A permanently installed 25mm thick steel casing vibro-driving was selected to advance the casing through the coarser gravels and into bedrock.

[4] This construction trestle was partially destroyed by severe flood conditions occurring 2 weeks *after* the conclusion of the load testing program



| L3 Depth (m) | L3 Soil Conditions | L4 Depth (m) | L4 Soil Conditions |
|---|---|---|---|
| 0-3 | Stiff Silt | 0-5.5 | Silt and Sandy Silt |
| 3-7.3 | Dense Gravely 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 Depth (m) | H3 Soil Conditions | H4 Depth (m) | H4 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 gravely 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 Eo (kPa) | Net Limit Pres. Pl* (kPa) | Ratio Eo/Pl* | Reload Modulus Er (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 | Gravely 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, Dc, 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.

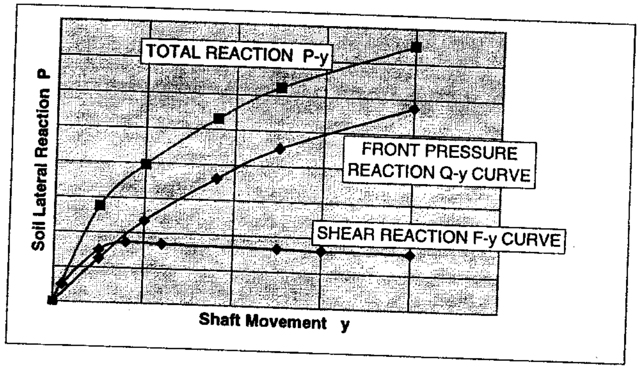



Figure 2 Front Pressure and Side Shear Components of the P-y Curve

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, Dc, 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, Es, is critical to predict movement of these shafts. Most theoretical work supports the conclusion that initial Es values are *not* functions of shaft diameter.

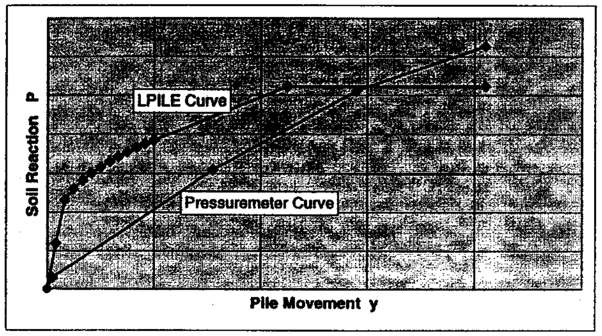



Figure 3 P-y Curves at 1.5 m Depth for H3 Shaft

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 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:

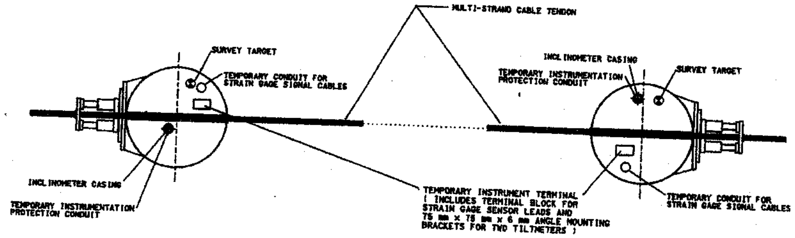
- Confirmation that first cycle deflections were correctly predicted by LPILE, and that the diameter effect are properly represented with PMT P-Y procedure.
- An understanding of pile behaviour under repeat, low frequency, lateral cyclic loads and high moments.
- Evaluation of possible creep under constant 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 multistrand 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.




PLAN

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, indinometers, survey points and load cells. Geokan 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 (½ 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 # of cycles | 2 | 4 | 5 | 3 | 2 |
| L3 & L4 # of cycles | 6 | 6 | 5 | 1* | 2 |

*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.

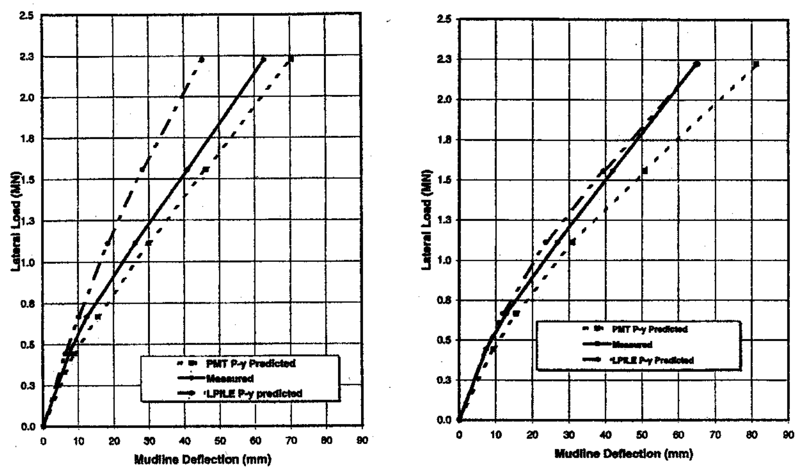



Figure 5: Measured and Predicted Load Deflection for L3 and L4

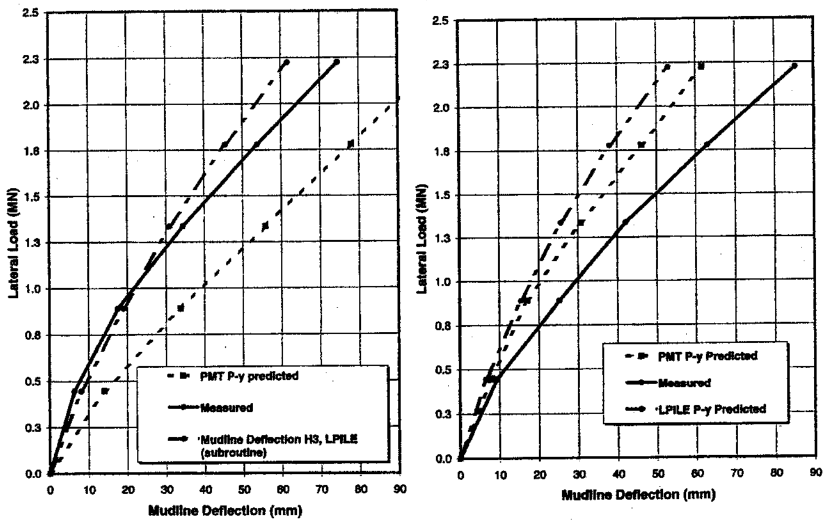



Figure 6: Measured and Predicted Load Deflection for H3 and H4

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 loosed 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:

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

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

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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

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## APPENDIX II : SYMBOLS

$y(1)$ = Measured deflection on the first (initial) cycle.

$y(N)$ = Deflection after N cycles

$N$ = Number of cycles

$\alpha$ = Power exponent

$E_s$= Pile Soil Modulus