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PERFORMANCE OF CAST-IN-PLACE PILES INSTALLED WITH A FIXED MAST DRILLING PLATFORM IN CLAYSTONE – GLENROCK WY

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The project team was tasked with the design and installation of a deep foundation system to support an expansion to a power generating station in Glenrock WY. Cast-in-place piles ***installed by a fixed mast drilling platform*** were assessed to be the most efficient foundation system for the subgrade conditions and range of design loads for the project. It was determined that conventional augered, cast-in-place piles could not be installed to the required pile toe elevations.

Subsurface materials consisted of interbedded coarse- and fine-grained soils to about 23 ft to 30 ft depth below the ground surface, underlain by carbonaceous shale (claystone) with interbedded sandstone. The overburden materials varied from very soft to medium stiff and loose to dense. The claystone was typically foliated and very weak to weak whereas the sandstone was typically massive and very weak to weak. The site characterization consisted of a variety of field and laboratory programs including soil borings with Standard Penetration Tests, pressure-meter testing, and geophysical testing.

A load test program was performed involving the compressive, lateral, and tensile testing of piles ranging from 14-in to 24-in diameter. Compression test piles were instrumented with vibrating wire strain gages.

The paper includes a discussion of the regional and local geologic and geotechnical parameters, particularly the results of the various in situ and laboratory test programs. The selected foundation system is introduced. Load test results are then presented along with analysis and discussion of these results with regard to performance of the piles in the claystone.

INTRODUCTION/PROJECT DETAILS

The project team designed and installed a deep foundation system for the expansion of an existing power generating station in Glenrock WY. The main components were a baghouse facility, lime storage facility, chimney, slurry pumps and fans.

The chimney was approximately 500 ft tall, 50 ft dia. with a two ft wall thickness. The lime day bin weighs about 575 kips, the slurry storage tank – 1,054 kips, the ash storage silo – 3,450 kips, and the recycle ash slurry storage tanks – 2,250 kips. Cuts and fills were less than three ft across the site. Planned foundation sizes ranged from 3-pile caps to mats with 120-plus supporting piles.

GEOLOGIC SETTING

The site is located within the Great Plains Physiographic. Province at the southern boundary of the Powder River Basin, just north of the tip of the Laramie Mountains (Figure 1). Bedrock consists of buff to light brown, thickly bedded sandstone; greenish gray, foliated marine shale; and bituminous coal beds of the Cretaceous Lance Formation. This formation was deposited during the very beginning of the Laramide Orogeny. The formation often contains conglomerate lenses which have been found during drilling at the power plant.

Portions of the power plant are overlying Quaternary North Platte River Alluvium, which includes sands and gravels derived from Precambrian basement rocks of the Southern

Rocky Mountains and Mesozioc sedimentary formations of the Wyoming Basin. The North Platte River initially formed on an aggraded surface of sedimentary and volcanic rocks within the Wyoming Basin, and later carved its way into the harder basement rocks after its course was established. The river is currently migrating to the south, and on its north bank, in the vicinity of the power plant, the sediments are typically loose and saturated due to their close proximity to the existing river bed.

Other overburden soils at the power plant and in the surrounding area are mainly derived from the sandstone beds within the Lance Formation. The formation is part of a large group of Cretaceous sedimentary formations in the Powder River Basin which contain the bulk of Wyoming's coal and uranium deposits, as well as large quantities of oil. Coal for the power plant is mined from an open pit mine in the Lance Formation near Glenrock.

SITE CHARACTERIZATION

Field Exploration:

A total of 11 test borings were performed during February 2008. The borings were advanced to approximate depths of 40 ft to 80 ft at the approximate locations shown on the Boring Location Diagram, Figure 2. The borings were advanced with a CME-55 drilling rig, utilizing 4¼-in OD-diameter, hollow-stem augers. The upper 6 ft to 8 ft of each boring was advanced using vacuum techniques to prevent utility line infrastructure damage.

A geophysical investigation was performed in the area of the stack, baghouse and SDA units in March 2008. The geophysical work included ReMi and refraction testing. There was good correlation between the ReMi models and the borehole data in terms of the soil / bedrock interface. Compression wave (P-wave) velocities in the upper zone of the overburden soils were on the order of 1,000 ft/sec while the P-wave velocities increased approximately five fold in the saturated zone of overburden soils. The shear wave velocities in the bedrock ranged from approximately 1,250 ft/sec to 1,750 ft/sec whereas the shear wave velocities in the overburden soils ranged from approximately 250 ft/sec to 1,000 ft/sec. The shear wave velocities averaged over the upper 100 feet (in accordance with the 2006 International Building Code) result in a Site Class "D".

To obtain complimentary subsurface information to support deep foundation recommendations, pressuremeter testing was performed in April, 2008. Two borings were drilled near test borings SB-01 and SB-06 and pressuremeter testing was completed using a BX sized probe. The pressuremeter test borings were advanced with a truck-mounted drilling rig, utilizing 3¼-inch-diameter, hollow-stem augers to provide hole support while advancing the pressuremeter test hole with mud rotary methods.

Laboratory Testing

Laboratory tests were conducted on selected soil and bedrock samples. The laboratory tests were performed in general accordance with applicable ASTM standards. Selected soil and bedrock samples were tested for the following engineering properties:

- Water Content
- Plasticity Index
- Grain Size
- Dry Density
- Consolidation
- pH
- Unconfined Compressive Strength
- Electrical Resistivity
- Swell/Consolidation
- Standard Proctor
- Water Soluble Sulfate / Chloride Content
- CBR

Subsurface Profile

A schematic of the subsurface conditions with SPT results is presented on Figures 3. The overburden soils consisted of clayey sand, silty sand and poorly graded sand with discontinuous layers of sandy silt, lean clay, and fat clay. The sands were loose to very dense in relative density and the silts and clays were very soft to medium stiff in consistency. The cleaner, poorly-graded sands typically consisted of medium- to coarse-grained particles with various amounts of gravel whereas the silty sands typically consisted of fine- to medium-grained particles. Typical grain size distribution plots for the overburden soils are presented in Figure 4.

The overburden clay, silt, and sand extended to the bedrock formation, which was encountered at depths ranging from 23 to 30 feet below grade. The bedrock consisted of claystone with interbedded sandstone lenses of the Cretaceous Lance Formation.

Bedrock varied from medium hard to very hard and generally increased in hardness with depth. Laboratory test results indicated the claystone bedrock possessed swell potentials of up to 7 percent with associated swell pressures of 1.5 ksf to 20 ksf. Unconfined compressive strengths of the rock materials ranged from 0.65 ksf to 1.5 ksf. The clay comprising the bedrock exhibited liquid limits ranging from 40 to 60 and plasticity indices ranging from 15 to 37.

Groundwater was encountered at depths of 4 to 10 feet below ground surface in the test borings at the time of field exploration. Subsequent monitoring revealed the static level at depths of 3 to 9 feet. The close correlation between the short term and stabilized water levels is a testimony to the nature of the granular overburden materials. A representative subsurface profile indicating the overburden, bedrock and groundwater conditions are presented in Figures 5.

SELECTION OF FOUNDATION TYPE

Based on the geotechnical conditions encountered in the test borings, the area of the proposed flue gas desulfurization project was underlain by approximately 23 to 30 feet of relatively weak, compressible sand, silt, and clay overburden materials over competent claystone bedrock. In addition to the compressible soil layers, the groundwater levels were relatively high, varying from about 3 to 9 feet below existing grade. Considering the size, type and loads for the various structures and the subsurface conditions, it was immediately determined that deep foundations should be used to support the various structures. The low strength and compressibility of the soils and the high groundwater precluded the use of shallow foundations for the major structures. It was determined that shallow foundations were feasible in some areas for small, light weight structures that were not settlement sensitive.

The project was initially budgeted considering 1600 drilled piers of 36-in diameter embedded approximately 10 ft into the claystone bedrock using temporary casing. For final design, several deep foundation alternatives, including drilled piers, driven piles and auger cast piles were initially evaluated by the design team. Given the saturated granular overburden soils the use of drilled piers was eliminated due to the necessity for casing during pier installation and the associated time delays and cost.

The level of design loading for several of the proposed process units ultimately eliminated the use driven piles as it was estimated that driven piles would not penetrate the competent claystone bedrock to sufficient depths to provide required tension loads.

Augered, cast-in-place piles were considered because of the potential to quickly install the piles through the saturated overburden and attain adequate bedrock penetration. After a comprehensive evaluation of available augercast pile installation techniques, the Augered Pressure-Grouted – Fixed Mast Crowded (APG-FMC) method was selected. The APG-FMC method incorporates a continuous-flighted, hollow-stem auger (similar to standard augercast piles) that is advanced by a drilling platform with a fixed mast instead of swinging leads (see Figure 6). A benefit of the fixed-mast platform is the potential to apply crowd pressure during the advancement of the auger, thus allowing rapid penetration into harder materials, such as the claystone bedrock. Reasonably large capacities are often generated, without the use of temporary or permanent casing.

It was anticipated that a similar number of much smaller diameter APG-FMC piles could be used in lieu of the drilled piers originally considered. The APG-FMC drilling platform also includes a data acquisition system for recording, displaying and reporting of drilling and grouting parameters during pile installation.

PREDICTED FOUNDATION BEHAVIOR FROM SITE CHARACTERIZATION

Based on the initial standard penetration tests performed, it was recognized that the claystone bedrock would be used to carry the structural loads via deep foundations. Using the Colorado SPT-based design method tempered with local experience, the end bearing and side friction capacity of deep foundations were estimated for preliminary design purposes. These preliminary parameters are outlined in Table 1.

Subsequent to the estimate of preliminary design parameters, pressuremeter testing was employed to further define the characteristics of the bedrock materials. The pressuremeter results (pressure vs. volume curves) of the four upper tests are shown in Figure 7. The derived results are summarized in Table 2. Unit shaft and toe resistances were estimated using the Menard method.

Table 1 – Preliminary Pile Design Parameters from SPT

Material	Allow. Side Friction [tsf]	Allow. End Bearing [tsf]
Overburden Soils	Neglect	NA
Bedrock, Upper 10 ft.	1	NA
Bedrock, Below 10 ft.	1.5	37.5

Table 2 – Summary of Pressuremeter Test Results and Derived Parameters

Boring	Depth [ft]	P_L [tsf]	E_m [tsf]	q_{all} [tsf] FS=3	f_s [tsf] FS=2
SB-01	32	74.5	862	49.4	2.1
SB-01	40	38	562	23.8	1.1
SB-01	60	46.5	878.5	33.6	1.3
		AVG	**720**	**35.6**	**1.5**
SB-06	35	31.5	474	38.5	0.9
SB-06	45	36.5	710	44.8	1.0
SB-06	65	59.5	1544.5	88	1.6
		AVG	**910**	**57.1**	**1.2**

Notes: P_L = Limit Pressure
E_m = Deformation Modulus
q = unit toe resistance
f_s = unit shaft resistance

Based on the pressuremeter test results the unit shaft and toe resistance parameters were revised, as shown in Table 3. The revised design parameters were in general agreement with the original estimates based on SPT data. Due to variability of the data in the upper zone of bedrock, it was determined that the lower bound values of the pressuremeter data would be used for preliminary design.

Because of the design loading conditions and the quantity of piles anticipated for the project, it was highly desirable to reduce the number of piles and reduce the size of individual piles. To maximize the structural capacity of the piles, particularly in compression, it was determined that the side friction within the bedrock, in addition to end bearing should be included in the design parameters.

The preliminary design, which was based on the parameters generated from the pressuremeter analysis, revealed that piles in the range of 18-in to 24-in diameter would provide sufficient load resistances.

Table 3 – Revised Pile Design Parameters from Pressuremeter

Material	Allow. Shaft Resistance Bedrock [tsf]	Allow. Toe Resistance Bedrock [tsf]
Overburden Soils	Neglect	NA
All Bedrock	1	32

FOUNDATION TEST PROGRAM DETAILS

A comprehensive test pile program was performed in April 2008. A total of nine test piles and 13 reaction piles were installed for the test program. The test piles were constructed with 14-in, 18-in, and 24-in diameters and overall lengths of approximately 31 ft to 33 ft.

Various data were recorded during installation including penetration rate, KDK pressure (the hydraulic fluid pressure applied to the drill stem) and Installation Effort (a measure of the energy the drilling platform expends during installation). An example of the data recorded (from the 14-in diameter compression test pile) is shown on Figure 8.

Piles were tested in general accordance with ASTM D 1143-81(94) for compressive loading, ASTM D 3689-90 for tensile loading, and ASTM D 3966-90 for lateral loading (free-head). Strain gages were installed within the compression test piles. Table 4 is a summary of the strain gage installation depths below the pile head at the ground surface.

Table 4 – Summary of Strain Gage Depths

	Diameter [in]		
	14	**18**	**24**
Depth [ft] of Strain Gage	5	5	5
	22.5	22.5	22.5
	27.5	27.5	27.5
	31	30	31

LOAD TEST RESULTS AND ANALYSIS

Compressive Capacity

Plots of applied compressive load and pile head displacement are presented in Figures 9 to 11 along with estimated ultimate load.

IBC 2006 allows for the evaluation of pile load tests with any of the following methods:

1. Davisson Offset Limit
2. Brinch-Hansen 90% Criterion
3. Butler Hoy Criterion
4. Other methods approved by the building official

Piles were evaluated by all of the methods above, including a method proposed by NeSmith (2002) whereby ultimate load is defined as the lesser of the following:

- The load at which the slope of the hyperbolic model of the pile head load-displacement relationship becomes 0.02 inches/ton
- The load at which the pile head deflection is equal to 6 per cent of the pile diameter

While the Davisson Offset Limit (DOL) method is listed as an acceptable process for evaluating ultimate load for pile foundations in IBC 2006, the method was originally developed for driven piles and is not appropriate for cast-in-place foundations (NeSmith and Siegel, 2009). Davisson (1993) recommends a modifier of between 2 and 6 when calculating the offset for evaluating a cast-in-place pile, as research on drilled piers has shown that toe deflections of 2 to 5 percent of the diameter are required to reach ultimate load, compared to less than 1% for driven piles. The DOL ultimate load was not considered in evaluating the performance of these test piles – it has been included on Figures 9 to 11 as a comparison only Table 5 shows estimations of ultimate load as described by NeSmith (2002), which is specific to augered or drilled cast-in-place foundations, as well as the Brinch-Hansen and Butler Hoy criteria.

Table 5 –Ultimate and Allowable Compressive Loads

Pile Dia. [in]	Ultimate Load [tons]			Allow. Load [tons]
	Brinch Hansen	Butler Hoy	NeSmith (2002)	
14	255	204	205	102
18	355	275	245	122
24	385	335	344	172

Note: Allowable load calculated by applying a factor of safety of 2.0 to the ultimate loads determined according to NeSmith (2002).

Compressive Load Transfer Within the Piles

From the installation records for the compressive test piles (see Figure 7), there is a clear increase in KDK Pressure as the drilling tool enters the claystone. Table 6 includes unit values for APG-FMC pile resistance.

The apparent depth below the test pile installation surface to the claystone is as follows:
14-in diameter: 24 ft
18-in diameter: 23 ft
24-in diameter: 23 ft

It is noted that the strain gage data from the lower two strain gages in the 18-in diameter APG test pile was considered unreliable. The unit loads for 18-in diameter APG piles in Table 6 have been interpolated based on the overall performance of the test pile as well as the data available from the other compression test piles. Figures 12 and 13 include interpreted load distribution curves for the 14-in and 24-in diameter piles.

Table 6 – Summary of Unit Compressive Capacity

	Unit Ult. Compressive Load [tsf]		
	14-in Dia.	**18-in Dia.**	**24-in Dia.**
Shaft in Soil	0.42	0.51	0.72
Shaft in Claystone	1.81	1.87	2.00
Toe in Claystone	117.58	67.91	40.74

Tensile Capacity

Ultimate load of the tension test piles was evaluated as the load at which the slope of the pile-head deflection vs. applied load curve reached 0.02 in/ton. The resulting ultimate loads, along with the measured deflection at an allowable load of one-half the ultimate load are presented in Table 7.

Table 7 – Tensile Resistance

Pile Diameter [in]	Ultimate Load [tons]	Allowable Load [tons] FS = 2	Deflection [in] at Allowable Load
14	74	37	0.012
18	107	53	0.023
24	180	90	0.095

Note: The deflections in Table 7 were well within the allowable deflection for production piles at this site.

Lateral Capacity

Ultimate loads, determined as the applied load at a deflection of 1 in, along with the measured deflection at an allowable load of ½ of the ultimate load are presented in Table 8.

Table 8 – Lateral Capacity

Pile Diameter [in]	Ultimate Load [tons]	Allowable Load [tons. FS = 2]	Deflection [in] at Allowable Load
14	21	10.5	0.32
18	19	9.5	0.18
24	27	13.5	0.18

Note: *The load at which the 14-in diameter pile was displaced by 0.25-in is estimated to be about 9 tons*

FINAL DESIGN PARAMETERS

The data from the test pile program was analyzed for end bearing and side friction in the bedrock and overburden soils. The strain gauge data indicated that there was a nominal side friction capacity within the overburden soils, which was to be expected. This capacity was conservatively discounted in the preliminary design due to strain incompatibilities of the overburden soils and harder bedrock into which the pile was end bearing. However, based on the field test data the side friction capacity in the overburden soils was included in the final design for economic reasons.

Because of the good correlation of results between the load test results and the pressuremeter data, it was determined that the final design values could be established using a factor of safety of 2 for end bearing and side friction. The final pile design values were ultimately based largely on the field test data with validation provided by the pressuremeter tests. The final values used for the compression design of the three pile sizes chosen to support the various process units throughout the project are presented in Table 9.

The recommended pile tension design values were evaluated based on the direct results of the pile load tests. The load test results indicated that the measured ultimate tensile load was very close to 80 percent of the measured side friction capacity in compression. Therefore, the final design value for tension was established as 80 percent of the recommended compressive resistance.

Table 9 - Pile Design Parameters - Compression

Pile Dia. [in]	Allow. Side Friction Overburden Soils [tsf]	Allow. Side Friction Bedrock [tsf]	Allow. End Bearing, [tsf]
14	0.25	0.95	40
18	0.25	0.95	37
24	0.25	0.95	30

The pile lengths required to provide the proposed design resistance for each pile size were estimated from Table 9. The estimated pile lengths to develop the design compression loads are outlined in Table 10.

Table 10 – Pile Embedment Lengths

Pile Dia. [in]	Design Comp. Load [tons]	Assumed Depth of Overburden Soils [ft]	Min. Embed. in Bedrock [ft]
14	75	23	5
18	125	23	9
24	220	23	16.5

Note Design load based on pile cross sectional area and concrete strength of 4 ksi.

The pile head movements for the various project structures, particularly the stack, were evaluated based on the final design parameters. For pile compression loading pile head movements were estimated to be on the order of 0.5 in or less. For tension loading and lateral loading the pile head movements were estimated to be less than 0.35 in.

SUMMARY AND CONCLUSIONS

The results of the pressuremeter tests, along with the initial field and laboratory program, indicated that a foundation system which could penetrate well into the bedrock while causing minimal harm to the state of the overlying materials would most efficiently support the proposed structures at this site. APG-FMC piles proved to be excellent in this regard. The data acquired on board the drilling platform during test and production pile installation provided additional value in the system.

There was also a very good correlation between the unit pile resistance values obtained from the load test program and the preliminary design estimates from the pressuremeter tests.
This provided a high degree of confidence and allowed the application of a very efficient final design.

The extent of the test pile program was considered appropriate considering the final number of foundation elements installed (~1,450). Also, the additional expenditure to perform the pressuremeter tests and the extensive load test program resulted in a substantial decrease in the final cost of the 1,450 18-in and 24-in diameter APG-FMC piles compared with the preliminary estimate of 1,600 36-in diameter drilled piers (based on the original field and laboratory programs alone).

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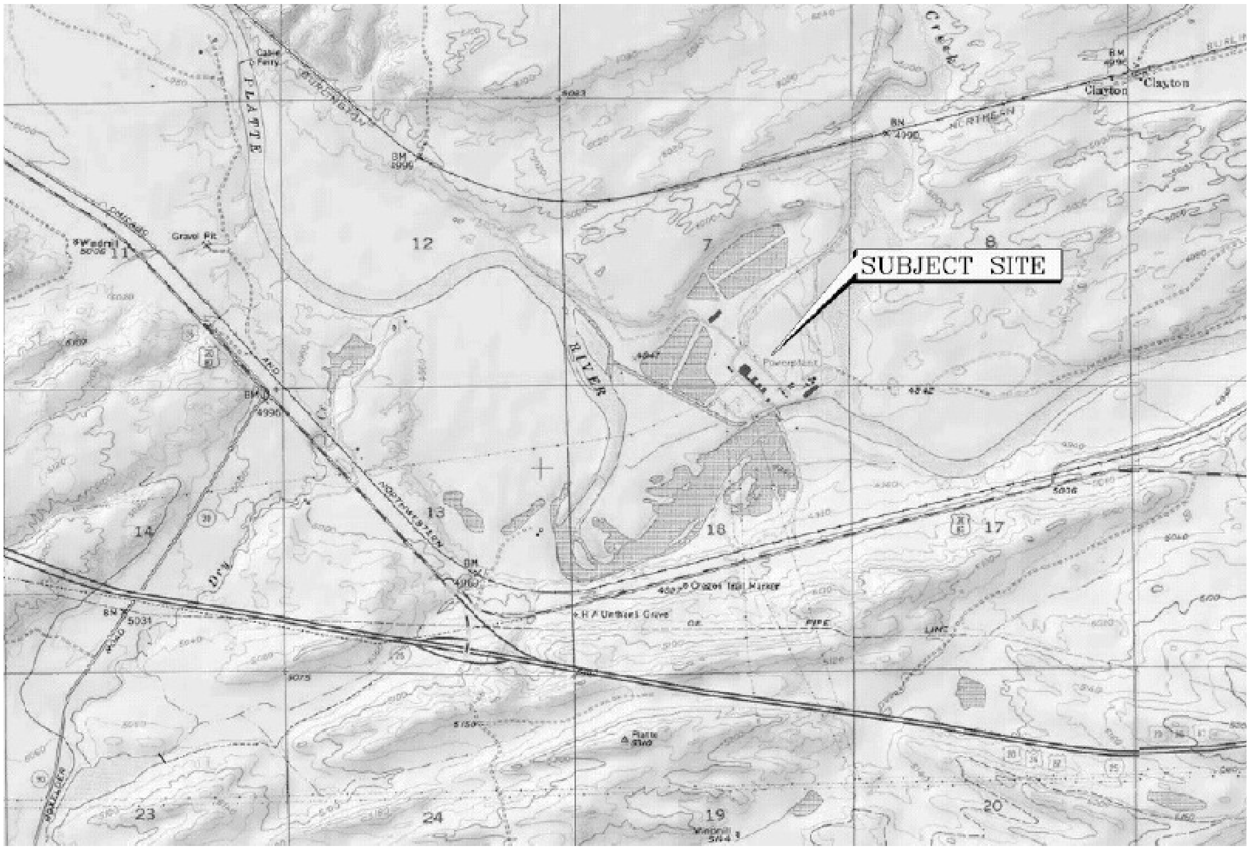

Figure 1 – Geologic Setting and Site Location

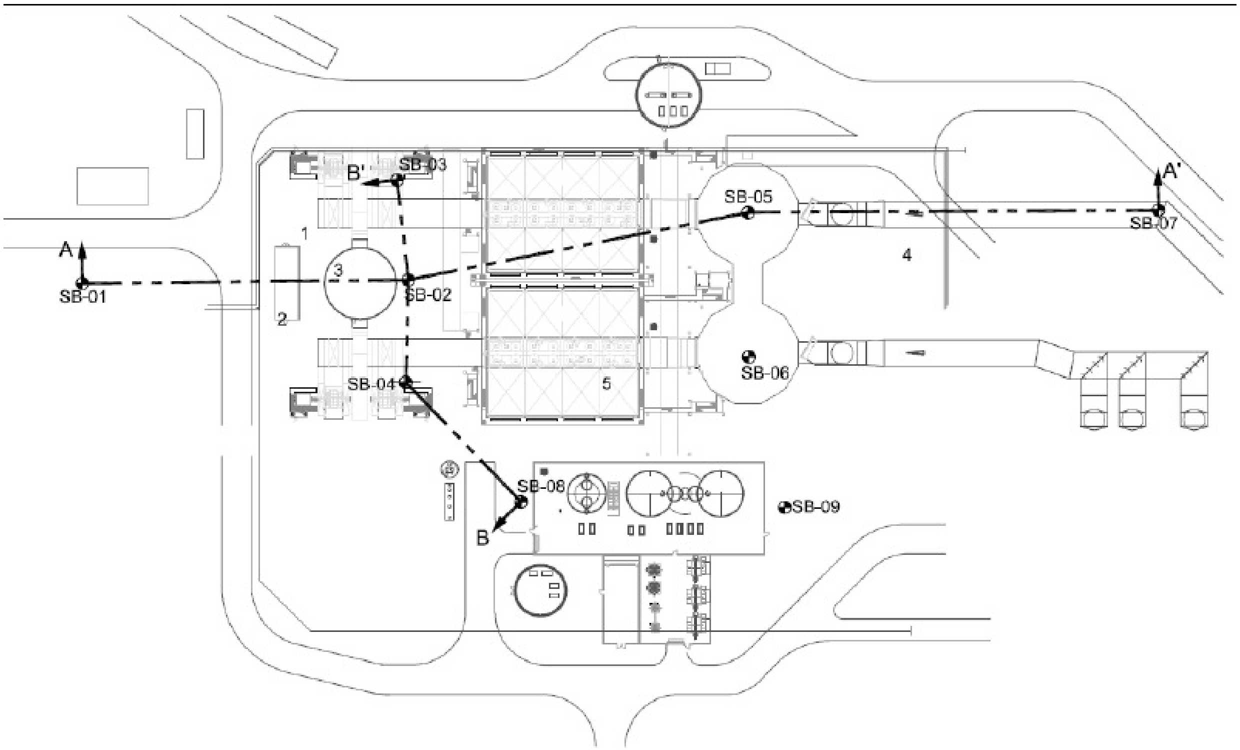

Figure 2 – Boring Location and Facility Plan

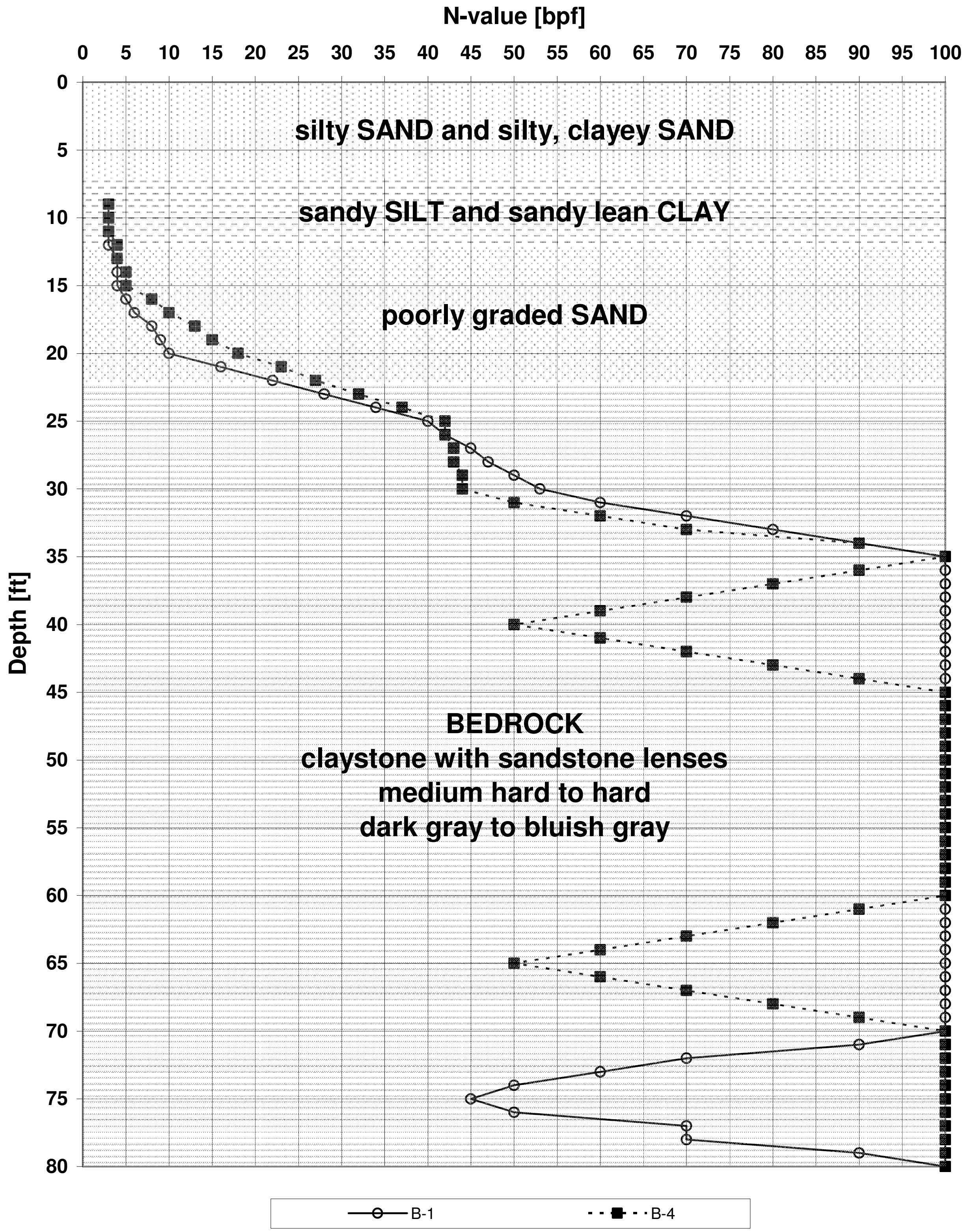

Figure 3 – Example Stratigraphy and SPT Results

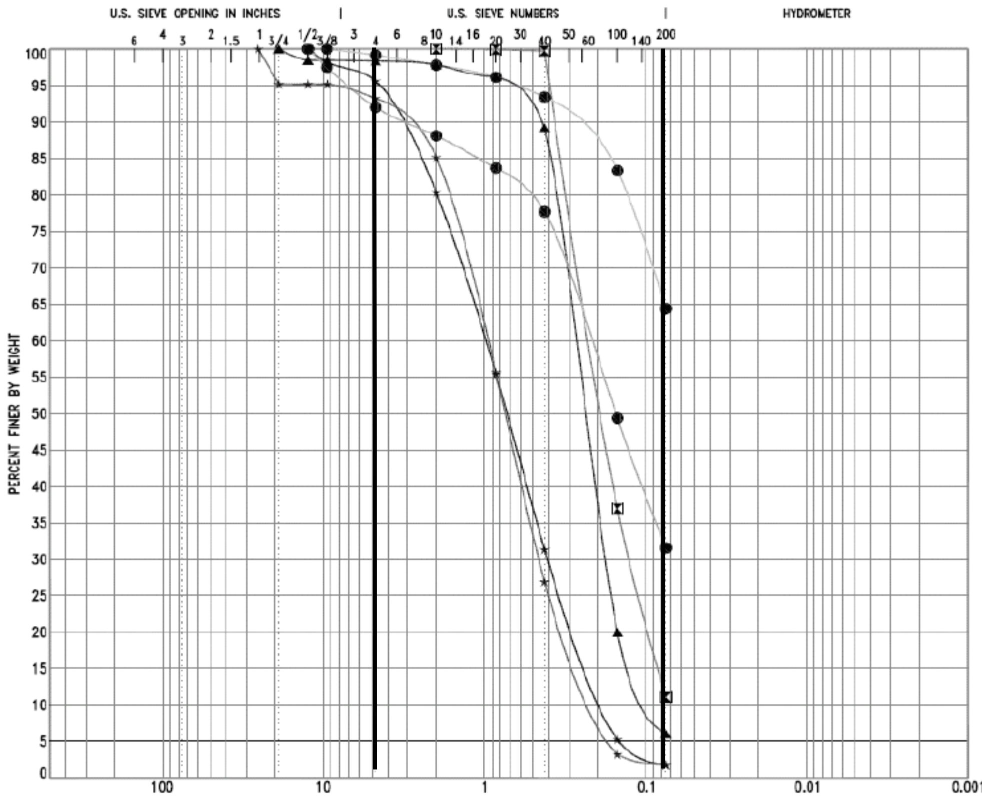

Figure 4 – Grain Size Distribution Plots for Overburden Soils

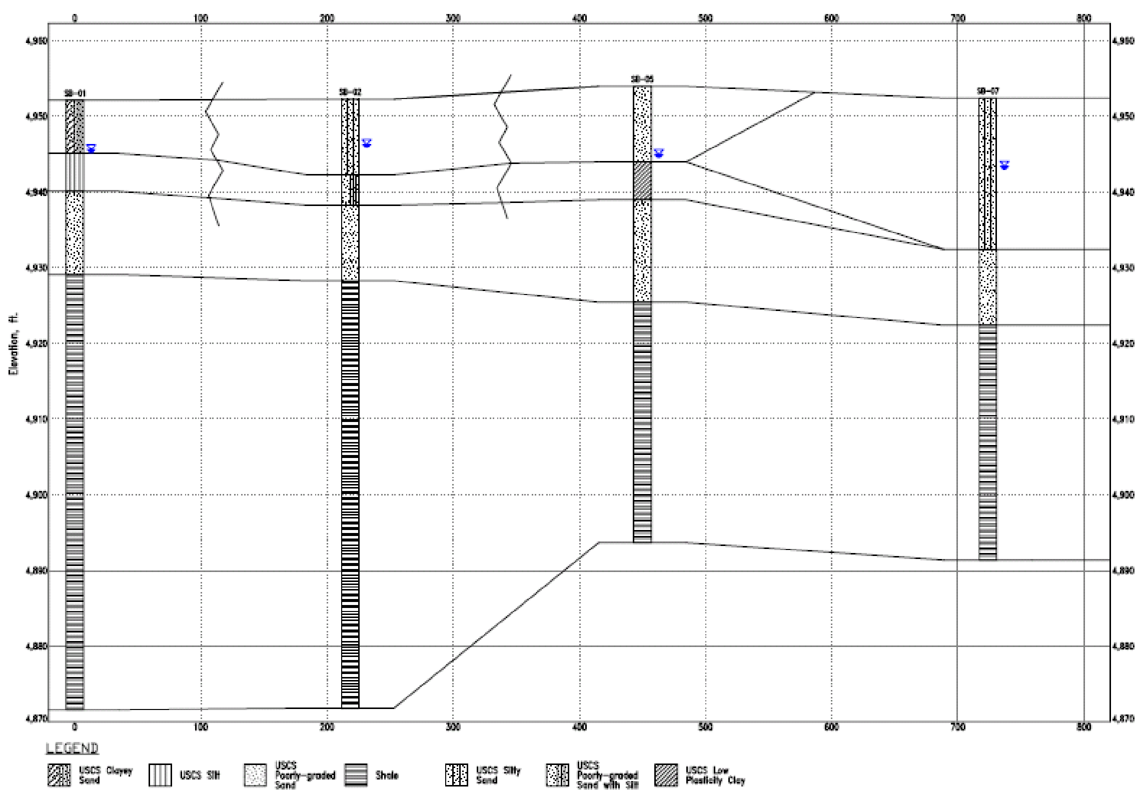

Figure 5 – Generalised Subsurface Profile

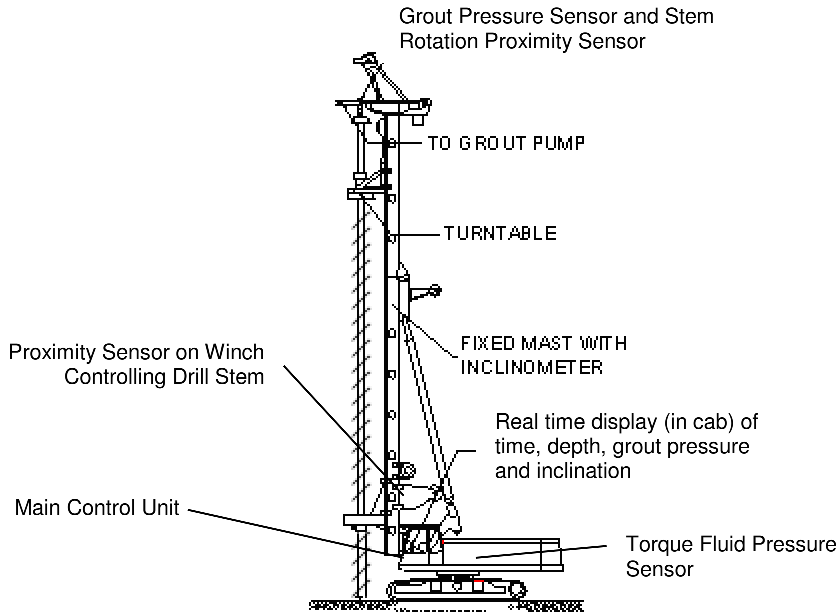

Figure 6 – Schematic of Drilling Platform and Generalization of Data Acquisition System

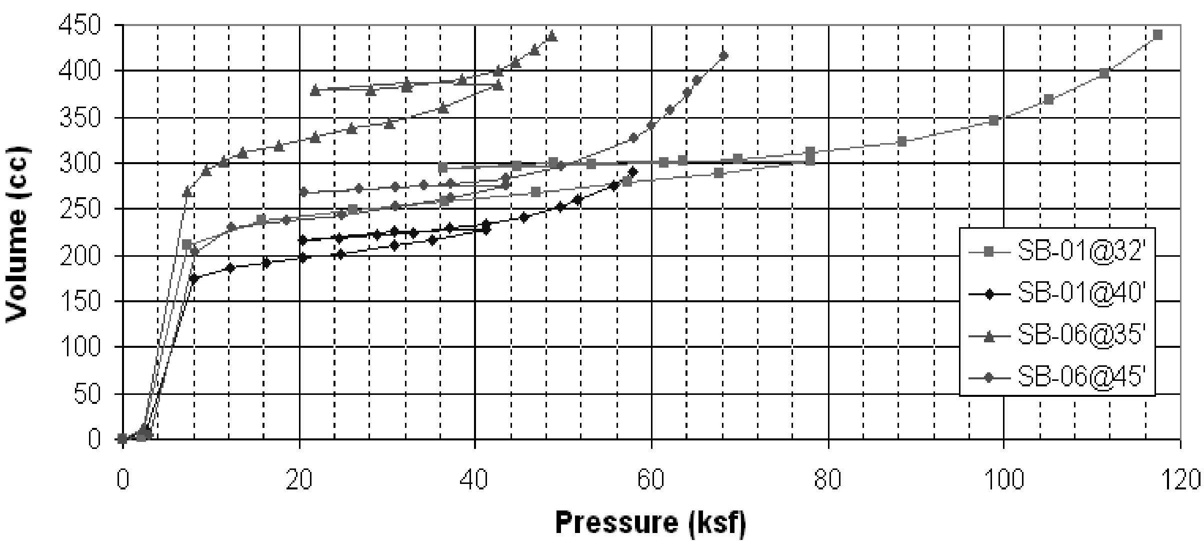

Figure 7 – Pressuremeter Test Results: Pressure vs. Volume Curves

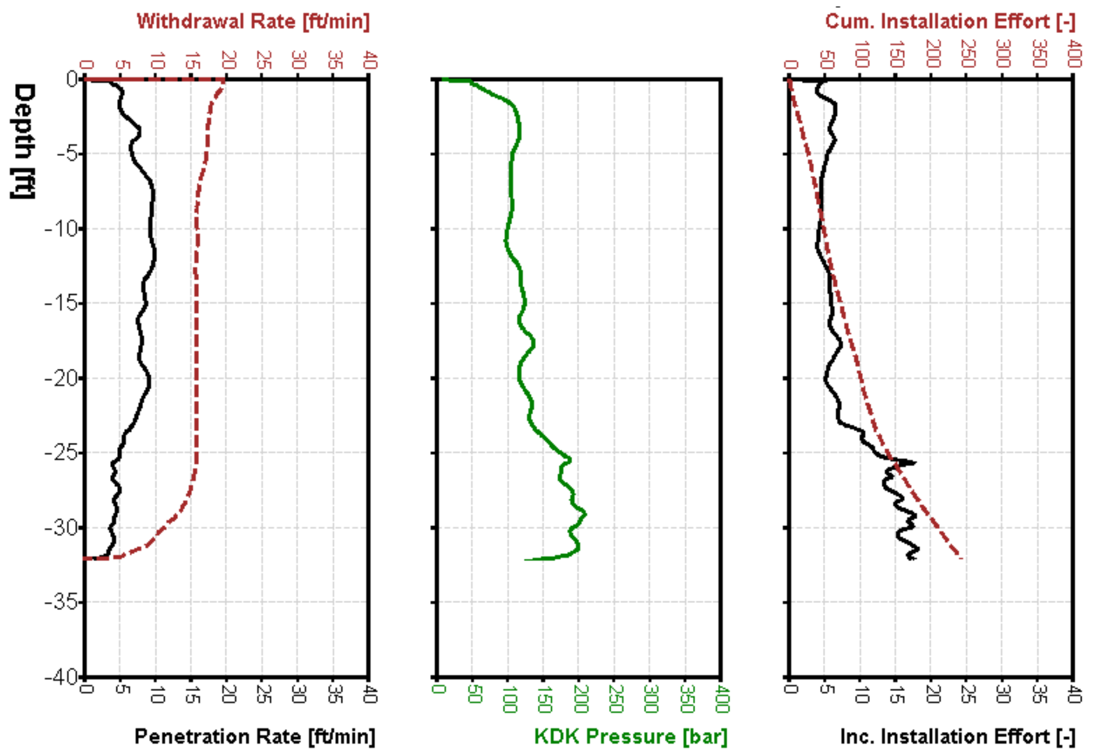

Figure 8 – Example Installation Parameters from 14-in APG-FMC Compression Test Pile

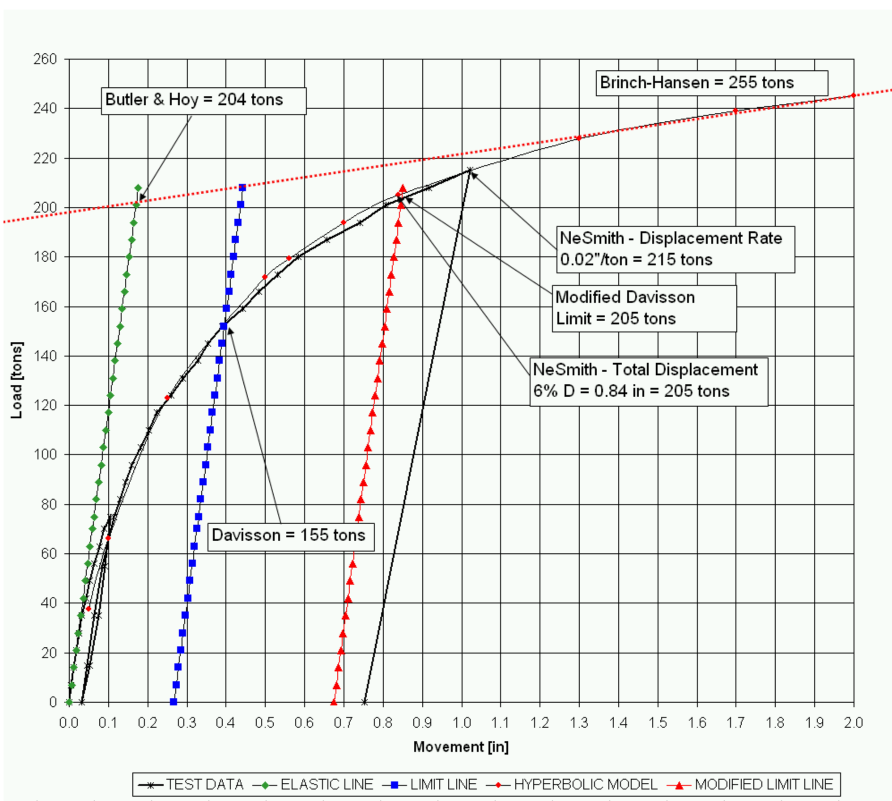

Figure 9 – Applied Loads vs. Pile Head Displacement
31-ft long – 14-in dia APG-FMC - Compression

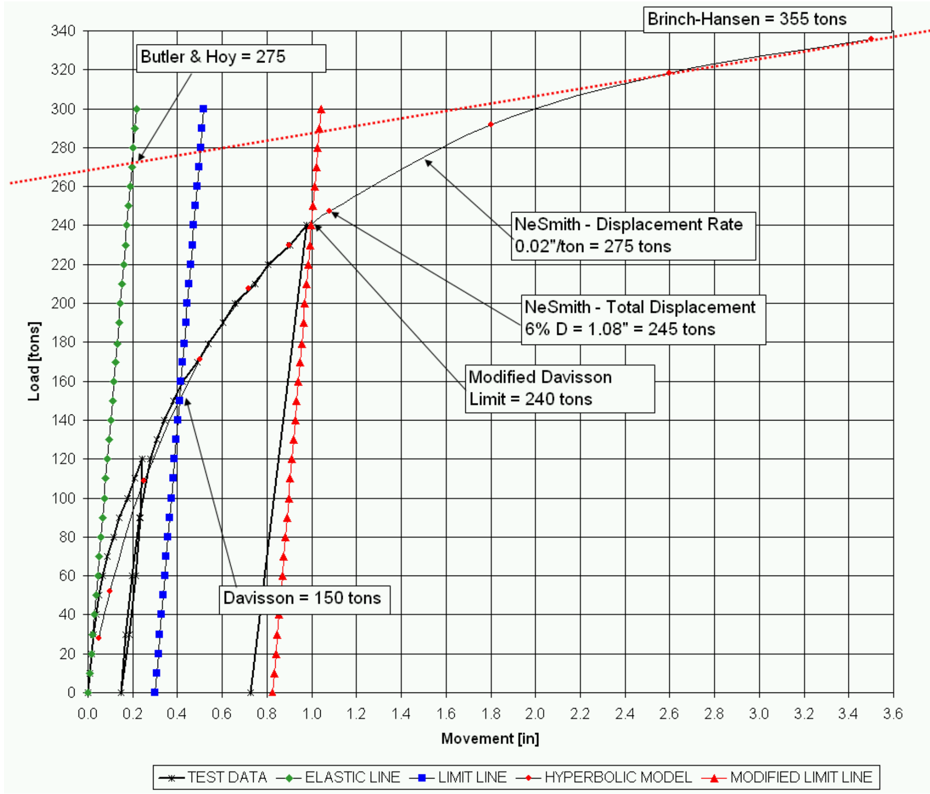

Figure 10 – Applied Loads vs. Pile Head Displacement
31-ft long – 18-in dia APG-FMC - Compression

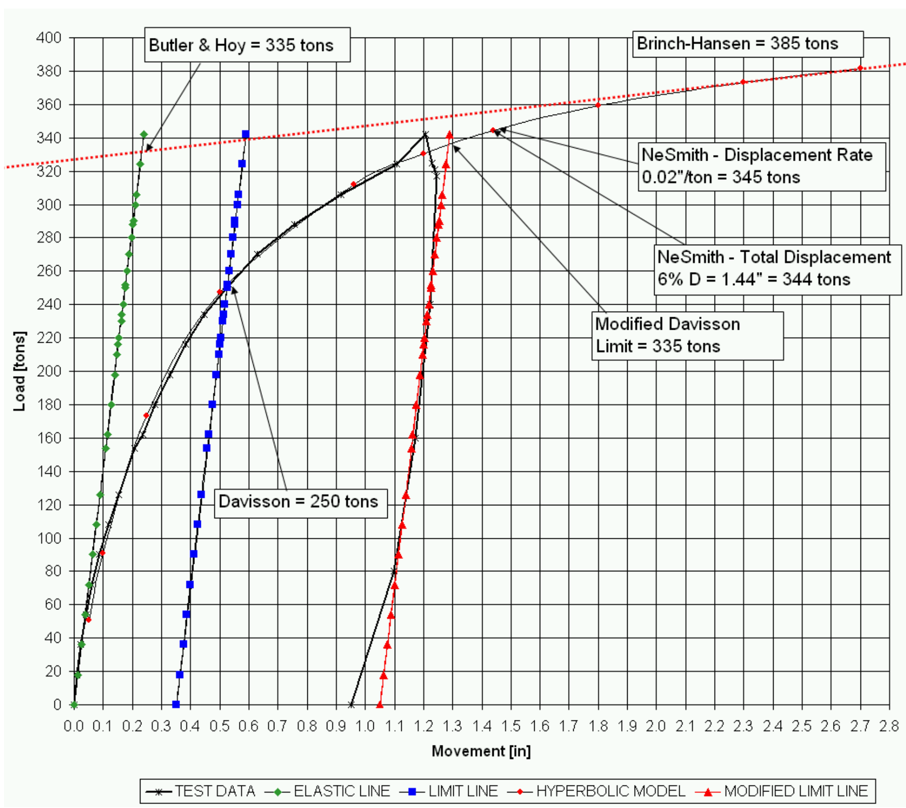

Figure 11 – Applied Loads vs. Pile Head Displacement
32-ft long – 24-in dia APG-FMC – Compression

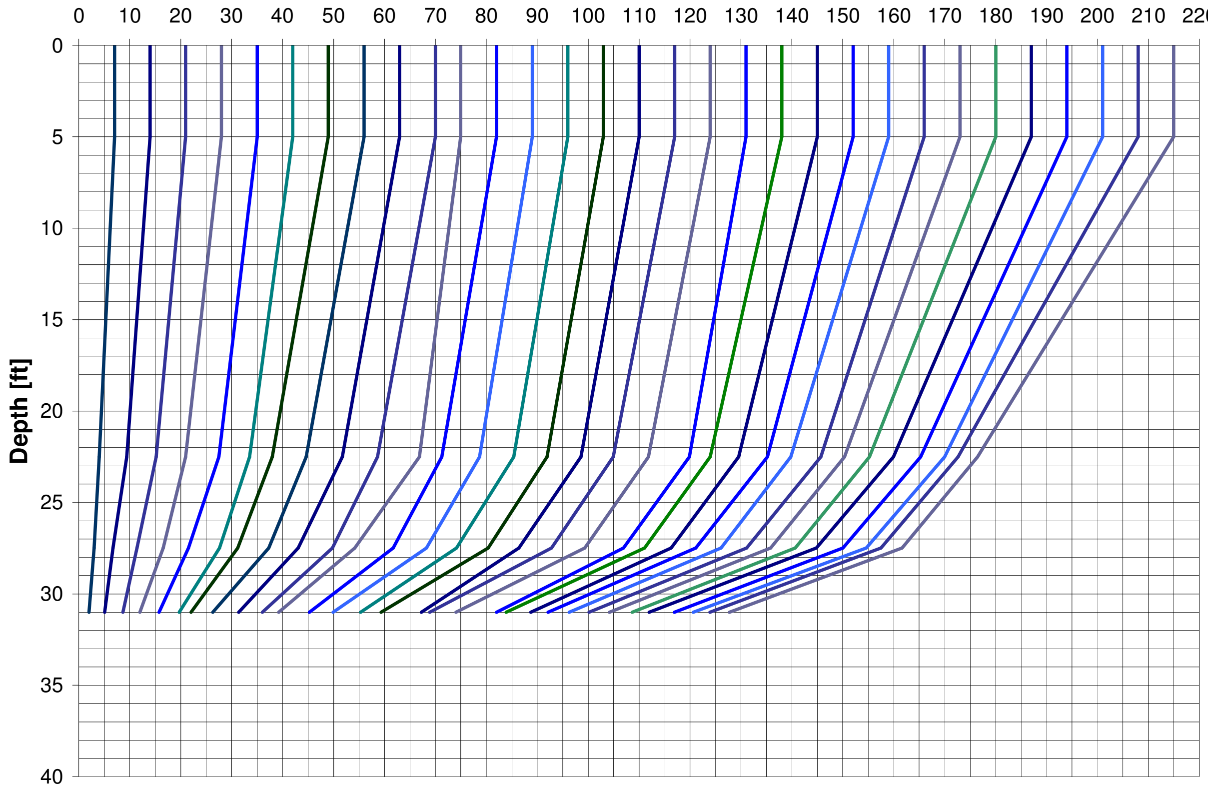

Figure 12 – Load Distribution During Load Testing
31-ft long – 14-in dia APG-FMC - Compression

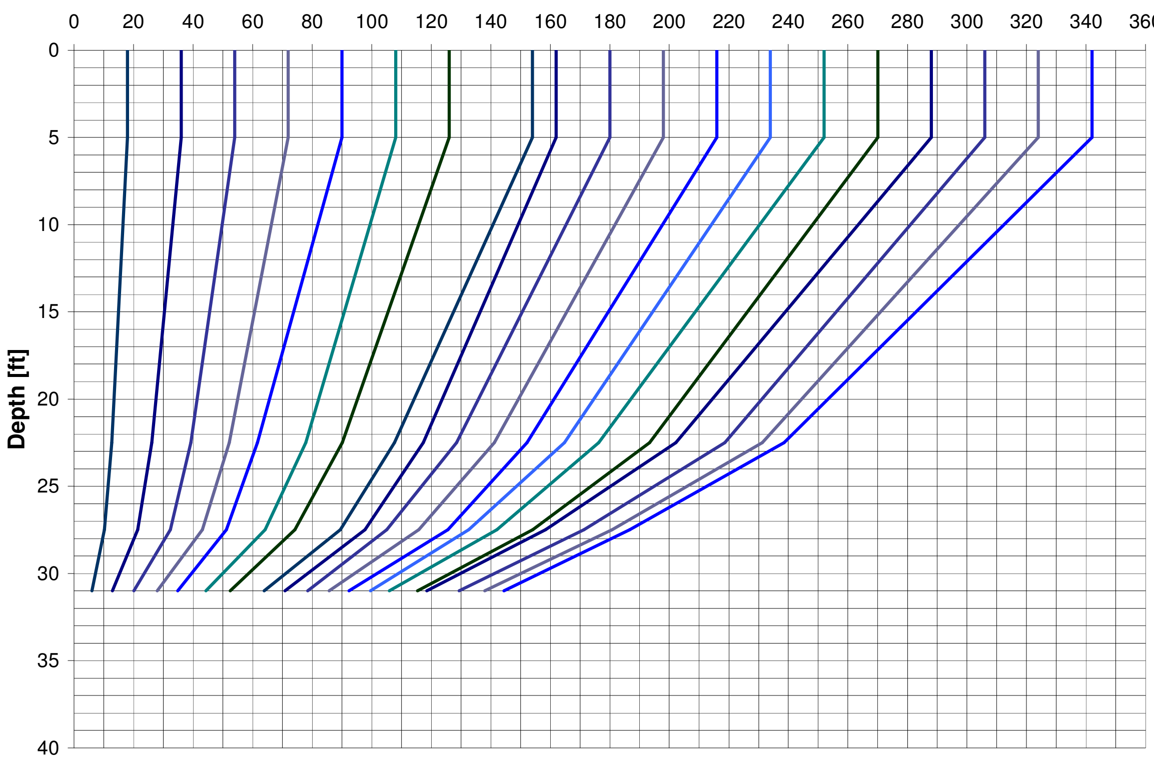

Figure 13 – Load Distribution During Load Testing
32-ft long – 24-in dia APG-FMC – Compression