CHAPTER 6

Response of Soil and Rock to Dynamic and Static Loading

The responses of soil and rock at the EGC ESP Site to dynamic and static loading were evaluated by conducting updated site-specific liquefaction potential evaluations and by drawing upon existing information in the CPS USAR for static loading conditions. Procedures for evaluating liquefaction potential have changed since the CPS Site investigation was conducted in the mid 1970s. In view of these changes, a new empirical method for evaluating liquefaction potential was used rather than drawing a comparison between conclusions reached for the CPS Site and what could occur at the EGC ESP Site because of similarities in site conditions. On the other hand, the methodologies that were used to evaluate site response to static loading (i.e., bearing capacity, settlement, and lateral earth pressures) have not changed since the CPS Site investigation and, therefore, information presented in the CPS USAR has been used to address these conditions. The foundation performance for the CPS Facilities has been good over the 20 years of operation, indicating that soil conditions are suitable for siting similar facilities in the area.

6.1 Liquefaction Potential

Liquefaction potential was evaluated for the EGC ESP Site based on the results of the geotechnical investigation conducted for the EGC ESP Site. The liquefaction evaluation was performed by the procedure recommended in Youd et al. (2001). This reference is a slightly updated version of the information presented in *Proceedings of the National Center for Earthquake Engineering Research (NCEER) Workshop on Evaluation of Liquefaction Resistance of Soils* (NCEER, 1997) and referenced in Draft Regulatory Guide 1105 (USNRC, 2001a).

6.1.1 Method of Evaluation

The liquefaction evaluation method calculates a FOS based on the expected soil shearing resistance and the expected maximum seismically-induced shearing stresses in a soil layer. Soil shearing resistance is quantified by the cyclic resistance ratio (CRR). Correlations have been developed to estimate the CRR from the SPT blowcount, with modifications for depth and SPT driving conditions (for example, hammer energy, driving efficiency, and soil sampler type). The expected shearing stresses induced by seismic loading are quantified by the cyclic stress ratio (CSR), which is the ratio of expected cyclic shearing stress to existing effective overburden. The CSR is proportional to the peak ground acceleration (pga) for the specified seismic loading. A magnitude scaling factor (MSF) is assigned based on the specified earthquake moment magnitude (M) expected to generate the specified pga. The FOS against liquefaction is calculated as:

$$FOS = \frac{CRR}{CSR}(MSF) \quad \text{Equation 6.1-1}$$

The MSF is smaller for larger M earthquakes (that is, reduces the FOS) to account for the longer duration of shaking and lower frequency vibrations typical of the larger events. The FOS against liquefaction is calculated for soil conditions at regular depth intervals to obtain a profile of FOS with depth.

The liquefaction procedure described by Yound et al. (2001) and presented in Draft Regulatory Guide 1105 (USNRC, 2001b) is appropriate for soils above a depth of approximately 75 ft. Below this depth, the potential for liquefaction is generally considered to be very low, except for very loose cohesionless soils. In addition, soils that are cohesive (that is, with USCS classification of CL or CH) or are located above the water table are not considered to be liquefiable, even at low calculated FOS. Silty soils (ML or MH) are not considered of concern for liquefaction unless they exhibit a certain combination of plasticity, in situ water content, and gradation (known as the Chinese criteria), even if the FOS is less than 1.0.

Subsurface conditions were modeled for each of the four EGC ESP Site borehole locations (that is, boreholes B-1 through B-4). The upper 75 ft of the subsurface was modeled in 5-ft depth intervals corresponding to the SPT blowcount and sample locations. Modeled information consists of the USCS soil classification, in situ unit weight, SPT blowcount, and estimated fines content (i.e., P200 fraction) for each depth interval. The SPT hammer efficiency was quantified by testing performed by GRL on August 2, 2002 (as described in Section 3.1.2.2). Depth to groundwater was modeled to be 6 ft bgs, based on nearby shallow piezometer results. This depth to groundwater represents a perched groundwater table at the top of the Wisconsinan till. The piezometric head drops with depth beneath the top of the Wisconsinan till, approaching nearly 30 ft bgs in the Illinoian till as indicated by piezometer results at B-1 Piezo. In most situations the degree of saturation associated with a perched condition is less than 100 percent, even when the water table is high. As the degree of saturation decreases, the resistance of a soil to liquefaction increases. This condition suggests that the static pore water pressures calculated in the liquefaction analyses, which are based on the modeled groundwater depth of 6 ft bgs, are therefore very conservative.

Liquefaction potential was evaluated for a peak ground acceleration of 0.3g over a range of earthquake magnitudes that could occur at the EGC ESP site. The pga value of 0.3g exceeds the design basis ground motion of 0.26g and therefore is conservative relative to the pga site characteristic. The pga of 0.3g was selected to be consistent with the value set forth in Regulatory Guide 1.60, which represented the peak acceptable value for the plants that form the basis of the Plant Parameters Envelope (see Section 1.4). It was reasoned that if the liquefaction potential were low at 0.3g, an additional margin of safety would exist for the site.

The maximum design earthquake **M** was derived by deaggregating the results from the probabilistic seismic hazard assessment (PSHA) for the EGC ESP Site. This information is summarized in Section 2.5 of the SSAR and discussed in detail in Appendix B to the SSAR. The three magnitudes used in the liquefaction analyses were as follows:

- M = 5.5 which is approximately equal to the mean magnitude assocated with a local source mechanism.

- **M = 6.5** which is approximately equal to the mean magnitude assocated with the Wabash source mechanism.
- **M = 8** which is an upper bound magnitude that might be assocated with a New Madrid source machanism.

Additional combinations of **M** and pga were also considered to illustrate the sensitivity of the calculated FOS to each parameter. One set of calculations was made for conditions at borehole B-1 by holding **M** constant at 6.5, with pga values of 0.25g, 0.30g, and 0.35g. Another set was made for constant pga of 0.25g, with **M** values of 6.75, 7.25, and 7.75.

Section 3.2 of Draft Regulatory Guide DG-1105 provides guidance on interpretation of liquefaction FOS for nuclear power plant sites. According to the guidance, soils with FOS less than 1.1 should be considered liquefiable at the specified earthquake loading. Soils with FOS greater than 1.4 are not considered to develop significant pore pressures during seismic loading. Although soils with FOS between 1.1 and 1.4 are not considered liquefiable, the effects of increased pore pressures on soil shear strength must be considered during design.

6.1.2 Results of Liquefaction Evaluations

Tables 6-1 through 6-4 and Figures 6-1 through 6-4 summarize the results of the liquefaction calculations for the range of earthquake loading and subsurface conditions modeled at boreholes B-1 through B-4, respectively. The FOS was calculated for each soil layer and for each of the modeled pga and **M** combinations. For soils that classify as silt (ML) via the USCS, but which pass the Chinese criteria, low calculated FOS does not indicate liquefaction potential because the soils are sufficiently cohesive. However, non-cohesive soils (silts and sands) with calculated FOS less than 1.1 are present at the four boreholes under a worse-case combination of pga and earthquake magnitude (that is, 0.3g for a **M** = 8 event). These soils are, therefore, considered potentially liquefiable. The results also indicate that additional soil layers from some boreholes have a FOS between 1.1 and 1.4, which indicates that pore pressures may generate during the maximum earthquake event. The potential for decrease effective stress in these soils will be considered for foundation design during the COL stage, although liquefaction is not anticipated to occur in these soils.

The potentially liquefiable non-cohesive soils at the EGC ESP Site (FOS less than 1.1) are all present within 60 ft of the ground surface. At each location, the granular soil is present in thin and possibly discontinuous zones within the Wisconsinan till or near the top of the Interglacial zone. Given the potential effects of founding structures on liquefiable soil deposits, which include loss in bearing support or seismic-related settlements, Category I nuclear facilities are not founded on liquefiable material. For the CPS Site, the upper 55 ft of soil were excavated. Although the material was removed primarily to limit settlements, it also provided a more liquefaction-resistant foundation. If a similar approach is taken for the EGC ESP Site, the liquefaction potential in the upper 60 ft can be avoided through selection and compaction of gravel backfill. Alternatively, some type of ground improvemant could be used to mitigate the potential for liquefaction. With several approaches available to address the liquefaction potential for the EGC ESP Site, this site characteristic is not be considered a constraining issue for siting.

Table 6-5 and Figures 6-5 and 6-6 illustrate the sensitivity of the calculated FOS to changes in **M** and pga. As shown, a reduction in pga from 0.35 to 0.25 increases the FOS by approximately 50 percent for each depth interval. A pga of 0.25 is essentially the same as the design basis earthquake, indicating that there is considerable reserve in terms of liquefaction resistance relative to the value or 0.3g used in the liquefaction evaluation. This observation suggests that the maximum excavation depth can be limited to 60 ft. Likewise, a unit reduction of earthquake magnitude (7.75 to 6.75) increases the FOS by approximately 50 percent. The smaller magnitude earthquake (M = 535 to 635 are more likely than the M> 7.75 event) are more likely, and these earthquakes have a lower potential for causing liquefaction. These results further support limiting the depth of excavation to 60 ft.

Based on the above information, the minimum site characteristic for liquefaction is absent below a depth of 60 ft bgs at the EGC ESP Site. Other seismic effects on soil pore water pressures and shear strengths at depths below 60 ft bgs can be managed with standard geotechnical practices, and should be considered in the design during the COL stage. However, these effects do not alter the suitability of the EGC ESP Site for construction of a reactor plant design.

6.2 Bearing Capacity

Ultimate bearing capacities for the CPS Facility were computed with conventional methods assuming a local shear failure condition, as described in Section 2.5.4.10.2 of the CPS USAR. The resulting ultimate bearing capacities for the Category I structures (except for the UHS) range from 39.9 to 60.6 tsf (79.8 to 121.2 kips per square foot [ksf]), as listed in Table 2.5-63 of the CPS USAR. A summary of structure foundation performance parameters for the CPS Facility structures is also provided in Table 6-6. Net foundation pressures for the Category I structures at the CPS Site are less than 2.5 tsf, resulting in FOS in bearing of greater than 20 for all structures except the containment structure, which has a FOS in bearing of 18.8.

Given the similarity in soil strengths, the ultimate bearing capacities of soils at the EGC ESP Site should be similar to values determined for the CPS Category I facilities, as long as the structures are of similar dimensions and are founded at similar or deeper depths. The foundation elevations for the CPS Facility structures range from 692 to 702 ft, except for the circulating water screen house and UHS outlet structure (located adjacent to Clinton Lake with lower foundation elevations), and the service building (not a Seismic Category I structure). Most of the foundations for the CPS Facilities were constructed over compacted select granular fill, and the depth of excavation for placement of this fill was approximately 55 ft bgs. The EGC ESP Site structure foundations may be constructed in a similar manner. Based on the above information, if foundation depths at the EGC ESP Site are similar or deeper than those at the CPS Site, the EGC ESP Site characteristic foundation soil bearing capacity will be significantly greater than 25 tsf.

Static moduli of subgrade reaction values at the CPS Site are also provided in the CPS USAR. The values range from 25 to 300 pounds per cubic in. As presented previously, soil classifications, strengths, and densities are consistent between the EGC ESP and CPS Sites. Based on this information, the site characteristic static moduli of subgrade reaction at the EGC ESP Site are similar to those at the CPS Site.

6.3 Settlement Potential

Predicted and actual foundation settlements for the CPS Facility structures are presented in Section 2.5.4.10.3 of the CPS USAR. Settlement criteria at the CPS Facility were achieved by excavating a 20-ft zone beneath the foundation level (roughly 800 ft by 800 ft in plan view) and replacing the excavated material with a compacted granular backfill. The soils below the excavated material, Illinoian and pre-Illinoian tills, are relatively incompressible as a result of overconsolidation caused by past glaciations. Some isolated pockets of sand at the base of the excavation that could not be compacted to meet density requirements were removed and replaced with a flyash backfill, as summarized in Section 2.6.3 of this Geotechnical Report.

Settlement was originally estimated for the CPS Facility to range from 1 to 2 in based on bearing pressures of 1 tsf to 2.5 tsf. These bearing pressures represented the net loads caused by the different Category I structures. Final settlements measured after construction were typically less than 0.5 in, suggesting that conditions were better than had been estimated for the CPS Site.

Results of the laboratory testing for the EGC ESP Site indicate that soil conditions are similar to those at the CPS Site, suggesting that settlements under the imposed bearing pressures will be small. As summarized in Section 5.2.3.1, values of C_c and C_r are consistent between the EGC ESP and CPS Sites. Values of P_c' from the test data for the EGC ESP Site are slightly lower than values from the CPS Site, but this would have no effect on settlements between the sites for foundation bearing pressures less than 5 tsf.

During the COL stage, facility-specific settlement analyses will need to be conducted to confirm that the structure foundation settlements will be acceptable. These analyses will consider the net bearing pressures applied by the structure, the foundation size, and the depth of the foundation. If the particular structure is located above the Illinoian till, it may be necessary to excavate to the top of the till and recompact select granular fill in the excavation to limit settlement, similar to what was done for the CPS Category I structures. Generally, the rate of settlement should be relatively fast because of the overconsolidated state of the till soils. These concerns will be addressed in the COL stage, but do not affect the suitability of the EGC ESP Site for construction of a reactor plant design.

6.4 Lateral Earth Pressures

Lateral earth pressures were determined for the CPS Facility structures for both static and seismic loading conditions, as summarized in Section 2.5.4.10.4 of the CPS USAR. At-rest pressures were used in the calculations because of the rigid behavior of the Category I walls. A static coefficient of lateral earth pressure of 0.47 was used for the analyses of the CPS Facility structures, corresponding to a typical friction angle of 32 degrees for the fine-grained soils at the excavation sidewalls. This was considered conservative, because compacted sand was actually placed within 40 ft of the foundation walls, which has a higher friction angle (38 degrees) and therefore lower static coefficient of lateral earth pressure (0.38). Dynamic water and soil pressure were also evaluated to account for seismic loading conditions.

The lateral earth pressures for the EGC ESP Site will depend on the selected reactor plant design. For those systems that are similar to the CPS Category I structures, the earth pressures will depend on the granular material used for backfill and the required compaction characteristics. The earth pressures for these structures can be accommodated during design of the walls for the facilities.

Some of the new reactor plant designs could involve embedment depths of over 100 ft. The soils at these depths consist of hard silts and clays. Earth pressures associated with these designs will have to be evaluated on the basis of the planned construction method. Special soil-structure interaction studies could also be required to evaluate the seismic performance of these deeply embedded structures. These concerns will be addressed in the COL stage, but do not affect the suitability of the EGC ESP Site for construction of a reactor plant design.

6.5 Other Considerations

Other geotechnical design issues have been identified which will be considered upon selection of the reactor plant design. These other considerations include design of slopes for the intake structure, performance of the UHS for the updated seismic hazard, and soil-structure interaction studies for the Category I structures. These concerns will be addressed in the design phase, but do not represent a constraining issue with regard to acceptability of the site.

A review was also performed to determine the location of dams occurring upstream and downstream of the EGC ESP Site. Results of this review concluded that there are no dams or other water retaining structures upstream of the facility that could result in inundation of the CPS or EGC ESP Sites if the water retaining structures were to fail for whatever reason. The main dam for the Clinton Lake is located approximately 3.5-mi downstream of the EGC ESP Site. It will not be modified as a result of EGC ESP Site development. The original design basis for the dam considered a much lower seismic ground motion than the safe shutdown earthquake (SSE), but is not relied upon for the safety of the CPS Facility. Only the CPS UHS was designed for the SSE. As noted previously, it will be necessary to confirm during the COL stage that the CPS UHS is capable of withstanding any higher levels of seismic-induced ground motions, if the selected reactor plant design must rely on the CPS UHS for emergency shutdown.

Another consideration will be dewatering of excavations during construction. The amount of dewatering will depend on the foundation elevation of the reactor plant design. During construction of the CPS Facilities, excavations extended 55 ft bgs. As reported in Section 2.5.4.5.1.3 of the CPS USAR, seepage into the foundation was very low in the natural clayey soils. However, more pervious sand layers and seams did contribute to the rate of seepage. According to the CPS USAR, dewatering was accomplished by a network of perforate metal pipe drains and ditches that collected the seepage at the periphery of the excavation. Similar procedures will likely be successful at the EGC ESP Site if the excavation elevations are similar. Construction dewatering for a reactor plant design that extends deeper than 55 ft will have to be evaluated during the COL stage if such a system is selected. Generally, dewatering should not be a significant construction issue due to the relatively low permeability of the soils at the EGC ESP Site.

Potential for seismically induced water waves (seiches) is considered in Sections 2.4.4 and 2.4.5 of the SSAR. Potential for effects of non-tectonic deformations is discussed in Section 2.5 of the SSAR. Based on the cited references, neither seiches nor non-tectonic deformations appear to be an issue at the EGC ESP Site.

CHAPTER 6

Tables

TABLE 6-1

Summary of Liquefaction Calculations - Expected Maximum Earthquakes - Borehole B-1

Design Parameters	
Ground Surface Elevation (ft msl):	738.6
Depth to Groundwater During Field Exploration (ft bgs):	6
Design Groundwater Depth (ft):	5
SPT Hammer Energy Ratio:	52
Liner Used in Sampler? {Yes or No}	No

					PGA = 0.35, M = 6.5			PGA = 0.25, M = 7.75		
Depth (ft bgs)	**Soil Type**	**Total Unit Weight (pcf)**	**SPT N-Value**	**Estimated Fines Content (%)**	**FOS**	**Liquefaction Concern?**	**Comments**	**FOS**	**Liquefaction Concern?**	**Comments**
3.5	CL	126	10	76	1.3	No	Clay Soil	1.1	No	Clay Soil
8.5	CL	126	13	76	1.1	No	Clay Soil	1.0	No	Clay Soil
13.5	CL	135	38	65	> 4	No		> 4	No	
18.5	CL	135	57	65	> 4	No		> 4	No	
23.5	SP	135	37	13	> 4	No		3.6	No	
28.5	SP	135	32	13	1.8	No		1.6	No	
33.5	CL	135	19	65	1.1	No	Clay Soil	1.0	No	Clay Soil
38.5	SW	135	13	13	**0.6**	**Yes**		**0.5**	**Yes**	
43.5	ML	128	40	57	> 4	No		> 4	No	
48.5	CL	128	14	57	0.8	No	Clay Soil	0.7	No	Clay Soil
53.5	ML	128	28	57	1.5	No		1.3	No	
58.5	ML	151	39	54	3.1	No		2.8	No	
63.5	ML	151	73	54	> 4	No		> 4	No	
68.5	ML	151	53	54	> 4	No		> 4	No	
73.5	ML	151	40	54	2.3	No		2.1	No	

TABLE 6-2

Summary of Liquefaction Calculations - Expected Maximum Earthquakes - Borehole B-2

Design Parameters	
Ground Surface Elevation (ft msl):	737.8
Depth to Groundwater During Field Exploration (ft bgs):	6
Design Groundwater Depth (ft bgs):	5
SPT Hammer Energy Ratio:	52
Liner Used in Sampler? {Yes or No}	No

					PGA = 0.35, M = 6.5			PGA = 0.25, M = 7.75		
Depth (ft bgs)	Soil Type	Total Unit Weight (pcf)	SPT N-Value	Estimated Fines Content (%)	FOS	Liquefaction Concern?	Comments	FOS	Liquefaction Concern?	Comments
3.5	CL	126	22	76	> 4	No		3.9	No	
8.5	CL	126	13	76	1.1	No	Clay Soil	1.0	No	Clay Soil
13.5	SM	135	100	13	> 4	No		> 4	No	
18.5	SM	135	100	13	> 4	No		> 4	No	
23.5	CL	135	40	65	> 4	No		> 4	No	
28.5	CL	135	29	65	2.9	No		2.6	No	
33.5	CL	135	40	65	> 4	No		> 4	No	
38.5	CL	135	41	65	> 4	No		> 4	No	
43.5	CL	128	13	57	0.8	No	Clay Soil	0.7	No	Clay Soil
48.5	CL	128	36	57	3.3	No		2.9	No	
53.5	ML	128	100	57	> 4	No		> 4	No	
58.5	ML	151	30	54	1.5	No		1.4	No	Clay Soil
63.5	SM	148	100	48	> 4	No		> 4	No	
68.5	SM	148	53	48	> 4	No		> 4	No	
73.5	CL	151	48	54	4.0	No		3.5	No	

TABLE 6-3

Summary of Liquefaction Calculations - Expected Maximum Earthquakes - Borehole B-3

Design Parameters	
Ground Surface Elevation (ft msl):	734.2
Depth to Groundwater During Field Exploration (ft bgs):	6
Design Groundwater Depth (ft bgs):	5
SPT Hammer Energy Ratio:	52
Liner Used in Sampler? {Yes or No}	No

					PGA = 0.35, M = 6.5			PGA = 0.25, M = 7.75		
Depth (ft bgs)	Soil Type	Total Unit Weight (pcf)	SPT N-Value	Estimated Fines Content (%)	FOS	Liquefaction Concern?	Comments	FOS	Liquefaction Concern?	Comments
3.5	CL	126	16	76	2.0	No		1.8	No	
8.5	CL	135	14	65	1.2	No	Clay Soil	1.1	No	Clay Soil
13.5	CL	135	31	65	> 4	No		> 4	No	
18.5	CL	135	28	65	3.2	No		2.8	No	
23.5	CL	135	16	65	1.1	No	Clay Soil	0.9	No	Clay Soil
28.5	CL	135	13	65	0.8	No	Clay Soil	0.7	No	Clay Soil
33.5	CL	135	32	65	3.7	No		3.3	No	
38.5	ML	128	12	57	**0.7**	**Yes**	Chinese Criteria OK	**0.7**	**Yes**	Chinese Criteria OK
43.5	SM	128	19	13	**0.8**	**Yes**		**0.7**	**Yes**	
48.5	SW	128	56	13	> 4	No		> 4	No	
53.5	ML	128	100	57	> 4	No		> 4	No	
58.5	SW	128	100	13	> 4	No		> 4	No	
63.5	ML	151	100	54	> 4	No		> 4	No	
68.5	ML	151	44	54	3.5	No		3.1	No	
73.5	ML	151	80	54	> 4	No		> 4	No	

Note: Soils that meet the Chinese Criteria are not considered liquefiable.

TABLE 6-4

Summary of Liquefaction Calculations - Expected Maximum Earthquakes - Borehole B-4

Design Parameters	
Ground Surface Elevation (ft msl):	735.4
Depth to Groundwater During Field Exploration (ft bgs):	6
Design Groundwater Depth (ft bgs):	5
SPT Hammer Energy Ratio:	52
Liner Used in Sampler? {Yes or No}	NO

					PGA = 0.35, M = 6.5			PGA = 0.25, M = 7.75		
Depth (ft bgs)	**Soil Type**	**Total Unit Weight (pcf)**	**SPT N-Value**	**Estimated Fines Content (%)**	**FOS**	**Liquefaction Concern?**	**Comments**	**FOS**	**Liquefaction Concern?**	**Comments**
3.5	CL	126	13	76	1.6	No		1.4	No	Clay Soil
8.5	CL	135	10	65	0.9	No	Clay Soil	0.8	No	Clay Soil
13.5	CL	135	9	65	0.8	No	Clay Soil	0.7	No	Clay Soil
18.5	CL	135	13	65	0.9	No	Clay Soil	0.8	No	Clay Soil
23.5	CL	135	80	65	> 4	No		> 4	No	
28.5	CL	135	22	65	1.4	No	Clay Soil	1.2	No	
33.5	CL	135	16	65	0.9	No	Clay Soil	0.8	No	Clay Soil
38.5	ML	128	27	57	1.9	No		1.7	No	
43.5	CL	128	19	57	1.0	No	Clay Soil	0.9	No	Clay Soil
48.5	SC	128	20	13	**0.8**	**YES**		**0.7**	**Yes**	
53.5	SM	128	54	13	> 4	No		> 4	No	
58.5	ML	151	28	54	1.4	No		1.2	No	
63.5	ML	151	32	54	1.6	No		1.4	No	
68.5	ML	151	33	54	1.6	No		1.4	No	
73.5	ML	151	44	54	3.0	No		2.6	No	

TABLE 6-5

Summary of Liquefaction Calculations - FOS Variation w/ M and PGA - Borehole B-1

Design Parameters	
Ground Surface Elevation (ft msl):	738.6
Depth to Groundwater During Field Exploration (ft bgs):	6
Design Groundwater Depth (ft bgs):	5
SPT Hammer Energy Ratio:	52
Liner Used in Sampler? {Yes or No}	No

					PGA = 0.25, M = 6.5			PGA = 0.30, M = 6.5			PGA = 0.35, M = 6.5		
Depth (ft bgs)	**Soil Type**	**Total Unit Weight (pcf)**	**SPT N-Value**	**Estimated Fines Content (%)**	**FOS**	**Liquefaction Concern?**	**Comments**	**FOS**	**Liquefaction Concern?**	**Comments**	**FOS**	**Liquefaction Concern?**	**Comments**
3.5	CL	126	10	76	1.8	No		1.5	No		1.3	No	Clay Soil
8.5	CL	126	13	76	1.6	No		1.3	No	Clay Soil	1.1	No	Clay Soil
13.5	CL	135	38	65	> 4	No		> 4	No		> 4	No	
18.5	CL	135	57	65	> 4	No		> 4	No		> 4	No	
23.5	SP	135	37	13	> 4	No		> 4	No		> 4	No	
28.5	SP	135	32	13	2.6	No		2.1	No		1.8	No	
33.5	CL	135	19	65	1.5	No		1.3	No	Clay Soil	1.1	No	Clay Soil
38.5	SW	135	13	13	**0.8**	**Yes**		**0.7**	**Yes**		**0.6**	**Yes**	
43.5	ML	128	40	57	> 4	No		> 4	No		> 4	No	
48.5	CL	128	14	57	1.1	No	Clay Soil	0.9	No	Clay Soil	0.8	No	Clay Soil
53.5	ML	128	28	57	2.0	No		1.7	No		1.5	No	
58.5	ML	151	39	54	> 4	No		3.6	No		3.1	No	
63.5	ML	151	73	54	> 4	No		> 4	No		> 4	No	
68.5	ML	151	53	54	> 4	No		> 4	No		> 4	No	
73.5	ML	151	40	54	3.2	No		2.7	No		2.3	No	

TABLE 6-5 (CONTINUED)

Summary of Liquefaction Calculations - FOS Variation w/ M and PGA - Borehole B-1

Design Parameters	
Ground Surface Elevation (ft msl):	738.6
Depth to Groundwater During Field Exploration (ft bgs):	6
Design Groundwater Depth (ft):	5
SPT Hammer Energy Ratio:	52
Liner Used in Sampler? {Yes or No}	No

					PGA = 0.25, M = 6.75			PGA = 0.25, M = 7.25			PGA = 0.25, M = 7.75		
Depth (ft bgs)	Soil Type	Total Unit Weight (pcf)	SPT N-Value	Estimated Fines Content (%)	FOS	Liquefaction Concern?	Comments	FOS	Liquefaction Concern?	Comments	FOS	Liquefaction Concern?	Comments
3.5	CL	126	10	76	1.6	No		1.3	No	Clay Soil	1.1	No	Clay Soil
8.5	CL	126	13	76	1.4	No	Clay Soil	1.2	No	Clay Soil	1.0	No	Clay Soil
13.5	CL	135	38	65	> 4	No		> 4	No		> 4	No	
18.5	CL	135	57	65	> 4	No		> 4	No		> 4	No	
23.5	SP	135	37	13	> 4	No		> 4	No		3.6	No	
28.5	SP	135	32	13	2.3	No		1.9	No		1.6	No	
33.5	CL	135	19	65	1.4	No	Clay Soil	1.2	No	Clay Soil	1.0	No	Clay Soil
38.5	SW	135	13	13	**0.7**	**Yes**		**0.6**	**Yes**		**0.5**	**Yes**	
43.5	ML	128	40	57	> 4	No		> 4	No		> 4	No	
48.5	CL	128	14	57	1.0	No	Clay Soil	0.8	No	Clay Soil	0.7	No	Clay Soil
53.5	ML	128	28	57	1.9	No		1.5	No		1.3	No	
58.5	ML	151	39	54	3.9	No		3.3	No		2.8	No	
63.5	ML	151	73	54	> 4	No		> 4	No		> 4	No	
68.5	ML	151	53	54	> 4	No		> 4	No		> 4	No	
73.5	ML	151	40	54	2.9	No		2.4	No		2.1	No	

TABLE 6-6

Summary of CPS Facility Structure Foundation Performance Parameters

Parameter	Range of Values	CPS USAR Citation
Foundation Elevation[a] (ft msl)	692 to 702	Section 2.5.4.10.2 Table 2.5-63
Net Static Foundation Pressure (ksf)[a]	2.3 to 4.8	Section 2.5.4.10.2 Table 2.5-63
Ultimate Bearing Capacity (ksf)[a]	79.8 to 121.2	Section 2.5.4.10.2 Table 2.5-63
Factor of Safety for Bearing Capacity[a]	18.8 to 50.5	Section 2.5.4.10.2 Table 2.5-63
Static Modulus of Subgrade Reaction (pci)	25 to 300	Section 2.5.4.10.2
Predicted Final Foundation Settlement (in)	1.0 to 1.7	Section 2.5.4.10.3 Table 2.5-67
Actual Final Foundation Settlement (in)[b]	-0.2 to 0.5	Section 2.5.4.10.3 Table 2.5-67
Static Coefficient of Lateral Earth Pressure[c]	0.47	Section 2.5.4.10.4

Notes:

a. Foundation elevations listed are for safety related structures in the upland area of the CPS Site.

b. Negative sign indicates swell

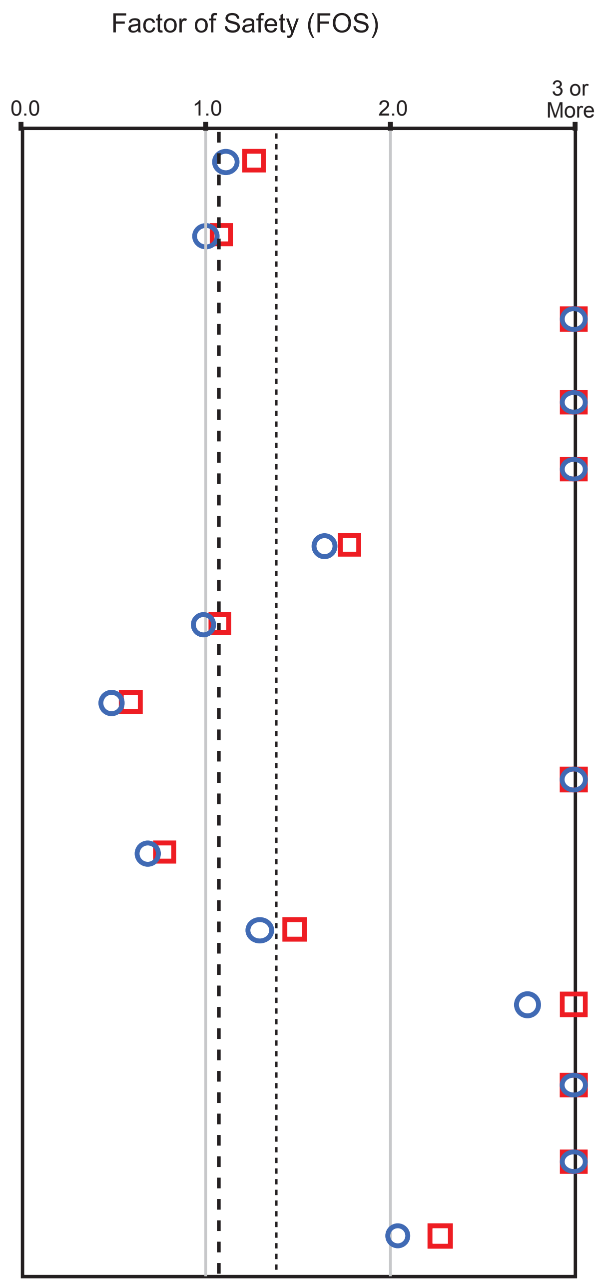
c. Corresponds to at-rest conditions in the fine-grained excavation sidewall soils (friction angle of 32 degrees).

Geotechnical Report for the EGC Early Site Permit

Figure 6-1

Factor of Safety Against Liquefaction with Depth Borehole B-1

Depth (ft)	USCS		Liquefaction Concern?
3.5	CL		No - Clay
8.5	CL		No - Clay
13.5	CL		No
18.5	CL		No
23.5	SP		No
28.5	SP		No
33.5	CL		No - Clay
38.5	SW		FOS < 1.1
43.5	ML		No
48.5	CL		No - Clay
53.5	ML		FOS < 1.4
58.5	ML		No
63.5	ML		No
68.5	ML		No
73.5	ML		No



Legend

= FOS < 1.1: Liquefaction possible

= FOS < 1.4: Seismic pore pressures possible

M=6.5, PGA=0.35

M=7.75, PGA=0.25

FOS = 1.1

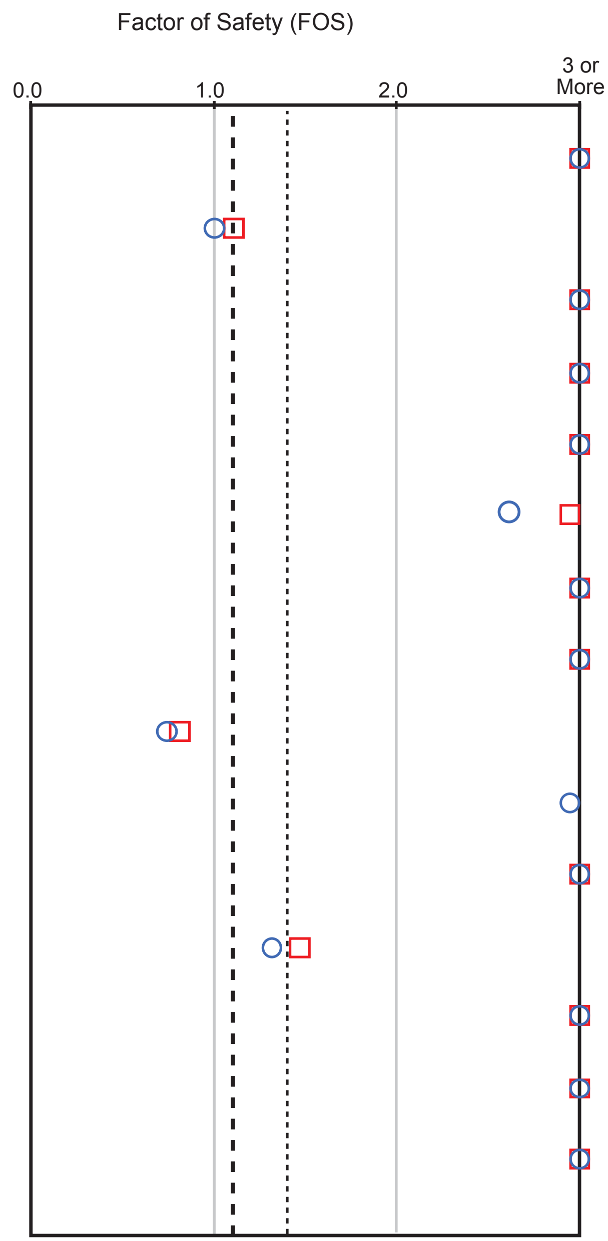
FOS = 1.4

Geotechnical Report for the EGC Early Site Permit

Figure 6-2

Factor of Safety Against Liquefaction with Depth Boring B-2

Depth (ft)	USCS		Liquefaction Concern?
3.5	CL		No
8.5	CL		No - Clay
13.5	SM		No
18.5	SM		No
23.5	CL		No
28.5	CL		No
33.5	CL		No
38.5	CL		No
43.5	CL		No - Clay
48.5	CL		No
53.5	ML		No
58.5	ML		FOS < 1.4
63.5	SM		No
68.5	SM		No
73.5	CL		No



Legend

= FOS < 1.1: Liquefaction possible

= FOS < 1.4: Seismic pore pressures possible

M=6.5, PGA=0.35

M=7.75, PGA=0.25

FOS = 1.1

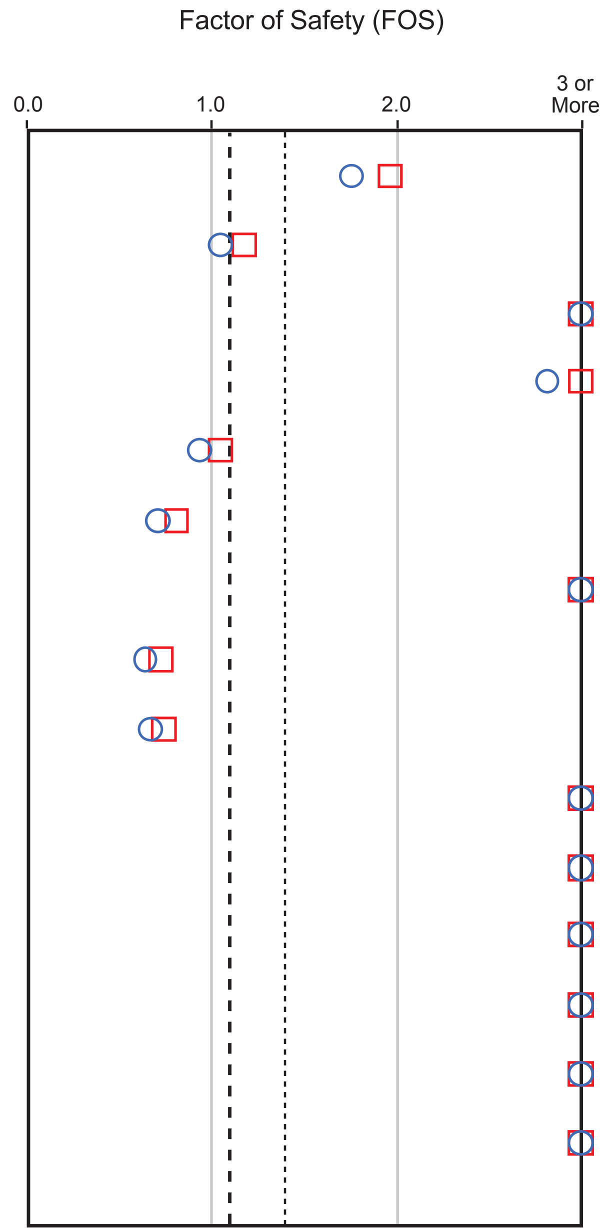
FOS = 1.4

Geotechnical Report for the EGC Early Site Permit

Figure 6-3

Factor of Safety Against Liquefaction with Depth Boring B-3

Depth (ft)	USCS		Liquefaction Concern?
3.5	CL		No
8.5	CL		FOS < 1.1
13.5	CL		No
18.5	CL		No
23.5	CL		No - Clay
28.5	CL		No - Clay
33.5	CL		No
38.5	ML		No - Chinese Criteria OK
43.5	SM		FOS < 1.1
48.5	SW		No
53.5	ML		No
58.5	SW		No
63.5	ML		No
68.5	ML		No
73.5	ML		No



Legend

= FOS < 1.1: Liquefaction possible

= FOS < 1.4: Seismic pore pressures possible

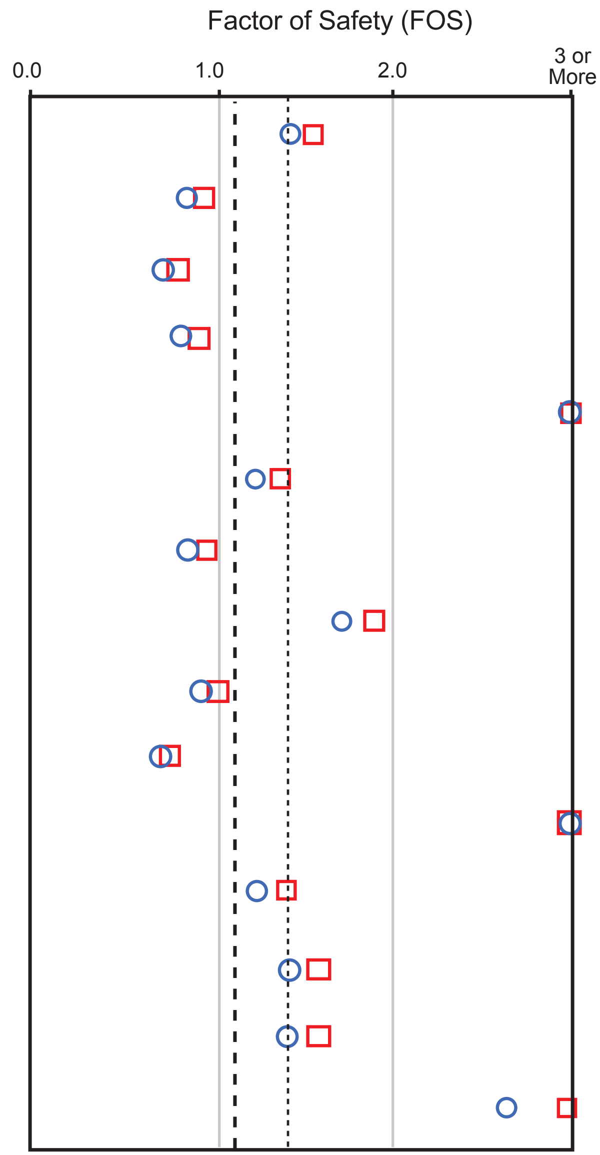
M=6.5, PGA=0.35

M=7.75, PGA=0.25

FOS = 1.1

FOS = 1.4

Depth (ft)	USCS		Liquefaction Concern?
3.5	C		No
8.5	C		No - Clay
13.5	C		No - Clay
18.5	C		No - Clay
23.5	C		No
28.5	C		No - Clay
33.5	C		No - Clay
38.5	ML		No
43.5	C		No - Clay
48.5	SC		FOS < 1.1
53.5	S		No
58.5	ML		FOS < 1.4
63.5	ML		FOS < 1.4
68.5	ML		FOS < 1.4
73.5	ML		No



Geotechnical Report for the EGC Early Site Permit

Figure 6-4

Factor of Safety Against Liquefaction with Depth: Boring B-4

Legend

= FOS < 1.1: Liquefaction possible

= FOS < 1.4: Seismic pore pressures possible

M=6.5, PGA=0.35

M=7.75, PGA=0.25

FOS = 1.1

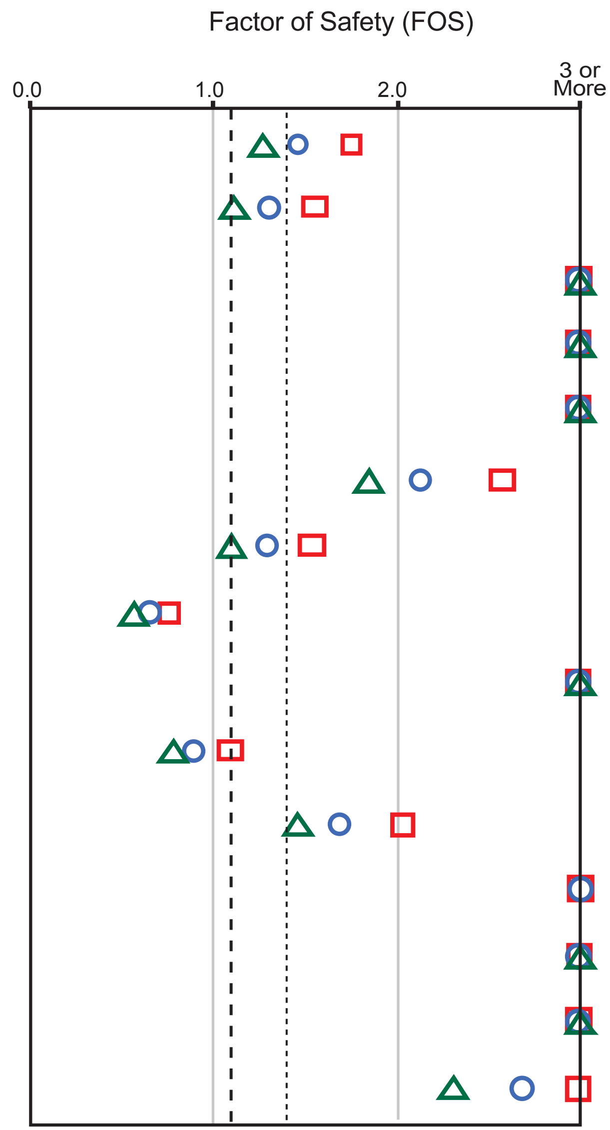
FOS = 1.4

Geotechnical Report for the EGC Early Site Permit

Figure 6-5

Factor of Safety Against Liquefaction with Depth: Variable PGA for M=6.5 at B-1

Depth (ft)	USCS		Liquefaction Concern?
3.5	CL		No - Clay
8.5	CL		No - Clay
13.5	CL		No
18.5	CL		No
23.5	SP		No
28.5	SP		No
33.5	CL		No - Clay
38.5	SW		FOS < 1.1
43.5	ML		No
48.5	CL		No - Clay
53.5	ML		No
58.5	ML		No
63.5	ML		No
68.5	ML		No
73.5	ML		No



Legend

= FOS < 1.1: Liquefaction possible

= FOS < 1.4: Seismic pore pressures possible

M=6.5, PGA=0.25

M=6.5, PGA=0.30

M=6.5, PGA=0.35

FOS = 1.1

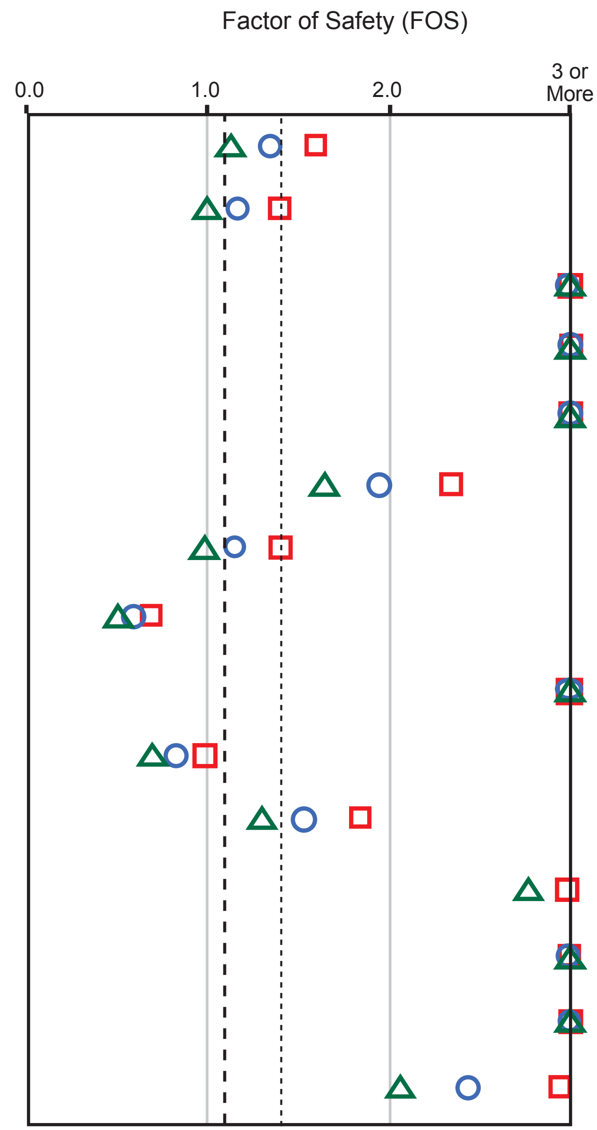
FOS = 1.4

Geotechnical Report for the EGC Early Site Permit

Figure 6-6

Factor of Safety Against Liquefaction with Depth: Variable M for PGA =0.25 at B-1

USCS	Depth (ft)		Liquefaction Concern?
3.5	CL		No - Clay
8.5	CL		No - Clay
13.5	CL		No
18.5	CL		No
23.5	SP		No
28.5	SP		No
33.5	CL		No - Clay
38.5	SW		FOS < 1.1
43.5	ML		No
48.5	CL		No - Clay
53.5	ML		FOS < 1.4
58.5	ML		No
63.5	ML		No
68.5	ML		No
73.5	ML		No



Legend

= FOS < 1.1: Liquefaction possible

= FOS < 1.4: Seismic pore pressures possible

M=6.75, PGA=0.25

M=7.25, PGA=0.25

M=7.75, PGA=0.25

FOS = 1.1

FOS = 1.4

CHAPTER 7

Conclusions Relative to Application for the EGC ESP

The geotechnical work described in this Geotechnical Report was performed to evaluate the suitability of the EGC ESP Site for the development of a new reactor plant design at some time in the future. The suitability of the site was evaluated on the basis of (1) whether any unacceptable geologic hazards exist and (2) whether geotechnical conditions will provide acceptable foundation support for a range of possible reactor plant designs. The information presented in this Geotechnical Report documents information that was used to decide on the suitability of the EGC ESP Site and serves as a basis for Section 2.5 of the SSAR and Section 3.6 of the ER for the EGC ESP Site. As discussed in the following two sections of this Geotechnical Report, the EGC ESP Site is considered suitable for future development of a reactor plant design from the standpoints of geology and geotechnical site characteristics. However, additional geotechnical work will be required at the COL stage to address reactor plant design-specific geotechnical design criteria.

7.1 Information for Early Site Permit Submittal

Geologic and geotechnical conditions at the EGC ESP Site are consistent with conditions at the CPS Site. The regional and local geologic conditions at the CPS and EGC ESP Sites are similar. No new geologic hazards were identified from EGC ESP Site investigation and reviews of the current literature. Comparisons of the soil layers and properties (e.g., soil classifications, strengths, compressibility, seismic velocities) are consistent between the CPS and EGC ESP Sites. The extensive database for the CPS Site, as summarized in the CPS USAR, and field explorations, laboratory testing, and engineering evaluations for the EGC ESP Site indicate the EGC ESP Site is suitable for development of a new reactor plant design. No new geologic hazards or geotechnical conditions were identified that could preclude successful construction and operation of a new reactor plant design within the EGC ESP Site footprint.

The minimum geotechnical site characteristics for the EGC ESP Site have been evaluated, and include the following:

- Allowable net bearing capacities exceed 25 tsf ;
- Liquefaction potential is absent if foundations extend to a depth of 60 ft bgs or greater, or if the material above 60 ft bgs is removed and replaced with compacted gravel fill or is improved; and
- Shear wave velocities below a depth of 50 ft bgs exceed 1,000 fps.

7.2 Information Required for Final Design

The geotechnical work completed for the EGC ESP Site is not considered sufficient for final design of the selected reactor plant design. Additional field explorations, laboratory testing, and engineering studies will be required. The extent of any additional explorations, laboratory testing, and engineering studies, if any are required, cannot be determined at this time. They will depend on the footprint and depth of the structures, the net weight, and the sensitivity of their performance to variations in soil properties. Regulatory Guide 1.132 will be utilized, together with foundation design requirements, to determine the locations, depths, and types of additional explorations. Nothing was identified during the geotechnical work described in this Geotechnical Report that would make any of these future investigations or studies particularly risky or difficult.

CHAPTER 8

References

American Society for Testing and Materials (ASTM). *Standard Test Method for Particle-Size Analysis of Soils*. ASTM D 422-63. 1998.

American Society for Testing and Materials (ASTM). *Standard Test Methods for Amount of Material in Soils Finer than the No. 200 (75μ) Sieve*. ASTM D 1140-00. 2000.

American Society for Testing and Materials (ASTM). *Standard Test Method for Penetration Resistance and Split Barrel Sampling of Soils*. ASTM D 1586-99. 1999.

American Society for Testing and Materials (ASTM). *Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes*. ASTM D 1587-00. 2000.

American Society for Testing and Materials (ASTM). *Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation*. ASTM D 2113-99. 1999.

American Society for Testing and Materials (ASTM). *Standard Test Method for Unconfined Compressive Strength of Cohesive Soil*. ASTM D 2166-00. 2000.

American Society for Testing and Materials (ASTM). *Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. ASTM D 2216-98. 1998.

American Society for Testing and Materials (ASTM). *Standard Test Method for One Dimensional Consolidation Properties of Soils*. ASTM D 2435-96. 1996.

American Society for Testing and Materials (ASTM). *Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)*. ASTM D 2487-00. 2000.

American Society for Testing and Materials (ASTM). *Standard Practice for Description and Identification of Soils (Visual – Manual Procedure)*. ASTM D 2488-00. 2000.

American Society for Testing and Materials (ASTM). *Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils*. ASTM D 2850-95. 1995.

American Society for Testing and Materials (ASTM). *Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils*. ASTM D 2974-00. 2000.

American Society for Testing and Materials (ASTM). *Standard Practice for Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction*. ASTM D 3740-01. 2001.

American Society for Testing and Materials (ASTM). *Standard Practice for Preserving and Transporting Soil Samples*. ASTM D 4220-95. 2000.

American Society for Testing and Materials (ASTM). *Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils*. ASTM D 4767-95. 1995.

American Society for Testing and Materials (ASTM). *Standard Practices for Preserving and Transporting Rock Core Samples*. ASTM D 5079-02. 2002.

Anderson, D.G. and K.H. Stokoe, "Shear Modulus: A Time-Dependent Soil Property." Dynamic Geotechnical Testing. American Society for Testing and Materials, STP 654, pp. 66-91. 1978.

Bauer, R.A., Trent, B.A., and P.B. DuMontelle. "Mine Subsidence in Illinois; Facts for Homeowners." *EG 144*. Illinois State Geological Survey. 1993.

Berggren, D. and C.S. Hunt**.** *A Guide to the Geology of the Farmer City Area, De Witt County, Illinois*. Illinois State Geological Survey. 1979.

Bergstrom, R.E., Piskin, K. and L.R. Follmer. "Geology for Planning in the Springfield-Decatur Region, Illinois." *Circular 497*. Illinois State Geological Survey. 1976.

Clinton Power Station (CPS). *Clinton Power Station Updated Safety Analysis Report (CPS USAR)*. Revision 10. January 2002.

Electric Power Research Institute (EPRI). *Guidelines for Determining Design Basis Ground Motions, Volume 1: Method and Guidelines for Estimating Earthquake Ground Motion in Eastern North America.* EPRI TR-102293, Project 3302. November 1993.

Huff, B.G. *Oil and Gas Developments in Illinois, 1987*. Illinois Petroleum. Vol 144. Illinois State Geological Survey. 1994.

Idriss, I.M. "Response of Soft Sites during Earthquakes," in J.M. Duncan, ed. *Proceedings, H. Bolton Seed Memorial Symposium*. BiTech Publishers. Vancouver, BC. Vol. 2. pp. 273-289. 1990.

Kempton, J.P. and B.L. Herzog. *Mapping the Mahomet Sand and Other Aquifers of De Witt and* Piatt Counties. OFS 1996-9. "Illinois Department of Natural Resources and the Illinois State Geological Survey". 1996.

Kempton, J. P., P.C. Heigold, and K. Cartwright. "Topography, Glacial Drift Stratigraphy, and Hydrogeology." Geological Society of America Special Paper 258. Mahomet Bedrock Valley, East-Central Illinois. 1991.

Killey, M.M., J.K. Hines, and P.B. DuMontelle. **"**Landslide Inventory of Illinois." *Circular 534*. Illinois Department of Energy and Natural Resources, State Geological Survey Division. 1985.

Meents, W.F. "Glacial-Drift Gas in Illinois." Circular 292. Illinois Department of Energy and Natural Resources, State Geological Survey Division. 1960.

National Center for Earthquake Engineering Research (NCEER). *Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils*. Technical Report NCEER-97-0022. 1997.

Panno, S.V. and K. C. Hackley. *The Origin of Weldon Springs. Open File Series 2001-3*. Illinois Department of Natural Resources and the Illinois State Geological Survey. 2001.

Panno, S. V., C.P. Weibel, and W. Li. Karst Regions of Illinois. OFS 1997-2 Illinois Department of Natural Resources and the Illinois State Geological Survey. 1997.

Schmertmann, J. H. *The Undisturbed Consolidation Behavior of Clay.* Transactions, ASCE. 1955.

Seed, H.B. and J.I. Sun. *Implications of Site Effects in the Mexico City Earthquake of Sept. 19, 1985 for Earthquake – Resistant Design Criteria in the San Francisco Bay Area of California*, Report No. UCB/EERC-89/103. Earthquake Engineering Research Center. University of California, Berkley. 1989.

Silva W.J., N. Abrahamson, G. Toro, and C. Costantino. "Description and Validation of the Stochastic Ground Motion Model." Report prepared by Pacific Engineering and Analysis, El Cerrito, CA for the Engineering Research and Applications Division, Department of Nuclear Energy, Brookhaven National Laboratory, Contract No. 770573. 1996.

Sun, J.J., R. Golesorkhi, and H.B. Seed. *Dynamic Moduli and Damping Ratios for Cohesive Soils*. Report No. EERC-88/15. Earthquake Engineering Research Center. University of California, Berkeley. 1988.

Treworgy, C.G., C.A. Hindman, L. Pan, and J.W. Baxter. *Evaluation of the Potential for Damage from Subsidence of Underground Mines in Illinois*. OFS 1989-2. Illinois State Geological Survey. 1989.

U.S. Nuclear Regulatory Commission (USNRC). *Design Response Spectra for Seismic Design of Nuclear Power Plants*. Regulatory Guide 1.60. 1973.

U.S. Nuclear Regulatory Commission (USNRC). *Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants*. Regulatory Guide 1.138. 1978.

U.S. Nuclear Regulatory Commission (USNRC). *Site Investigations for Foundations of Nuclear Power Plants*. Regulatory Guide 1.132. 1979.

U.S. Nuclear Regulatory Commission (USNRC). *Identification and Characterization of Seismic Sources and Determination of Safe Shutdown Earthquake Ground Motion*. Regulatory Guide 1.165. 1997.

U.S. Nuclear Regulatory Commission (USNRC). *Site Investigations for Foundations of Nuclear Power Plants*. Draft Regulatory Guide 1101 (proposed revised Regulatory Guide 1.132). 2001a.

U.S. Nuclear Regulatory Commission (USNRC). *Procedures and Criteria for Assessing Seismic Soil Liquefaction at Nuclear Power Plant Sites*. Draft Regulatory Guide 1105. 2001b.

U.S. Nuclear Regulatory Commission (USNRC). *Laboratory Investigations of Soils and Rocks for Engineering Analysis and Design of Nuclear Power Plants.* Draft Regulatory Guide 1109 (DG-1109) (proposed Revision 1 of Regulatory Guide 1.138). 2001c.

Vucetic , M. and R. Dobry. *Effect of Soil Plasticity on Cyclic Response. Journal of Geotechnical Engineering*. American Society of Civil Engineers. Vol. 117, No. 1. pp. 89-107. 1991.

Weibel, C. P. and S.V. Panno. *Karst Terrains and Carbonate Rocks in Illinois*. Illinois State Geological Survey. 1997.

Youd, T.L. et al. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils."

Journal of Geotechnical and Geoenvironmental Engineering. American Society of Civil Engineers. Vol. 127, No. 10. pp. 817-833. 2001.

Attachment A-1
Borehole Logs and Rock Coring Logs

PROJECT NUMBER: 171881.S1.02.01

BORING NUMBER: B-1

Sheet: 1 of 4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-1; Southwest corner of ESP footprint

ELEVATION: 738.6 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bio-Bore Drilling Mud

WATER LEVELS: 17.4' bgs on 8/28/2002 **START:** 7/22/2002 **FINISH:** 7/23/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
0.0					Ground Surface	6" Dia. Steel Casing Set to 5 feet bgs	
	1-SS	SS	1.3'	6-5-5-6	Sandy CLAY (CL), sl. moist. Yellowish Brown (10YR, 5/4). Some small gravel and fine sand.	TOP OF LOESS	
	2-ST	ST	2.0'	NA	Bottom of Tube: As above.	Pock. Pen.: 2.5, 2.25 TSF Torvane: 6.0 TSF	GR
	3-SS	SS	1.5'	6-6-7	As above. Becomes brown (10YR, 5/3) at 9.6'.		
10.0	4-ST	ST	1.4'	NA	As above.	Pock. Pen: >4, >4 TSF Torvane: 8.0 TSF	
	5-SS	SS	1.0'	16-17-21	Clay (CL), dry to sl. moist. Dark grey (2.5Y, 4/1). Some small gravel.	TOP OF WISCONSINAN TILL	
	6-PIT	PIT	2.2'	NA	As above.	Pock. Pen: Not measured (sample loose in tube)	GR
	7-SS	SS	1.5'	17-25-32	As above, some gravel to 1" dia.		
20.0							
	8-SS	SS	0.8'	15-17-20	Top 0.2': Fine SAND (SP), moist. Dark grey (10YR, 4/1). Bottom 0.6': CLAY (CL), as above (14-15').		
	9-SS	SS	1.2'	8-15-17	As above (CLAY (CL)). 1/2" thick fine sand seam @ 29'.		
30.0							

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-1

Sheet: 2 of 4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-1; Southwest corner of ESP footprint

ELEVATION: 738.6 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bio-Bore Drilling Mud

WATER LEVELS: 17.4' bgs on 8/28/2002 **START:** 7/22/2002 **FINISH:** 7/23/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	10-SS	SS	1.0'	9-9-10	CLAY (CL), as above.		
	11-PIT	PIT	1.8'	NA	Top of Tube: As above. Bottom of Tube: SAND (SW). Dark Grey (2.5Y, 4/1).	Pock. Pen: Not measured (sand sample)	
	12-SS	SS	1.2'	5-5-8	CLAY (CL), moist. Very dark grey (10YR, 3/1). Plastic.	TOP OF INTERGLACIAL ZONE	
40.0	13-ST	ST	1.3'	NA	Bottom of Tube: CLAY (CL), as above (14-15').		GR
	14-SS	SS	0'	16-18-22	No Recovery.		
	15-SS	SS	0.8'	15-17-20	SILT (ML), moist. Dark grey (10YR, 4/1). Some small gravel & sand, more than above (14-15').		
	16-SS	SS	1.5'	5-6-8	Lean CLAY (CL), moist. Grey (10YR, 4/1). Slightly more plastic.		
50.0	17-ST	ST	2.0'	NA	Bottom of Tube: As above.	Pock. Pen: 1.5, 1.5 TSF Torvane: 5.0 TSF	GR
	18-SS	SS	1.5'	9-13-15	SILT (ML), moist. Dark greenish grey (Gley1, 4/10GY). Sandy from 54.0-54.2'.		
	19-ST	ST	0.9'	NA	Bottom of Tube: As above.	Pock. Pen: >4, >4 TSF Torvane: 8.0 TSF	
60.0	20-SS	SS	1.5'	15-16-23	Sandy SILT (ML), sl. moist. Grey (10YR, 5/1). Some sand and small gravel.	TOP OF ILLINOIAN TILL	

REV4

PROJECT NUMBER: 171881.S1.02.01	BORING NUMBER: B-1
	Sheet: 3 of 4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-1; Southwest corner of ESP footprint
ELEVATION: 738.6 **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bio-Bore Drilling Mud
WATER LEVELS: 17.4' bgs on 8/28/2002 **START:** 7/22/2002 **FINISH:** 7/23/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	21-PIT	PIT	1.4'	NA	Bottom of Tube: As above.	Pock. Pen: >4, >4 TSF Torvane: >10 TSF	
	22-SS	SS	1.2'	21-32-41	As above.		
	23-SS	SS	1.2	16-22-31	As above.		
70.0							
	24-SS	SS	1.2	8-16-24	Top 0.6': SILT (ML), sl. moist. Dark reddish grey (5YR, 4/2). Organic - possibly peat. Bottom 0.6': SILT (ML), moist. Dark grey (10YR 4/1). Slightly plastic.		
	25-PIT	PIT	1.7'	NA	Bottom of Tube: As above, some small gravel and sand.	Pock. Pen: 3.5, 2.75 TSF Torvane: 6 TSF	
	26-SS	SS	1.2'	17-22-25	Sandy SILT (ML), sl. moist. Dark Grey (10YR, 4/1). Some sand and small gravel.		
80.0							
	27-SS	SS	0'	61-50\4"	No recovery.		
	28-PIT	PIT	1.0'	NA	Bottom of Tube: As above. large cobble (2" dia., rounded) black basalt, partially blocked end of tube.	Pock. Pen & Torvane not measured (obstruction)	
	29-SS	SS	0.4'	100\5"	As above.		
90.0							

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-1

Sheet: 4 of 4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-1; Southwest corner of ESP footprint

ELEVATION: 738.6 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bio-Bore Drilling Mud

WATER LEVELS: 17.4' bgs on 8/28/2002 **START:** 7/22/2002 **FINISH:** 7/23/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE				SOIL DESCRIPTION			COMMENTS	
	NUMBER	TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	30-PIT	PIT	2.6'	NA			Bottom of Tube: As above.	Pock. Pen: >4, >4 TSF Torvane: >10 TSF	GR, C, UU
	31-SS	SS	0.5'	51-50\3"			As above.		
								End of Boring at 100' BGS. Piezometer B-1 constructed within borehole (see separate log)	
100.0	32-SS	SS	0.3'	100\3"			As above.		
							End of Boring		
110.0									
120.0									

REV4

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-2

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint

ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
0.0							Ground Surface	6" dia. steel casing set to 5 feet bgs.	
	1-SS	SS	1.5'	7-12-10			Sandy CLAY (CL), moist. Light olive brown (2.5Y, 5/4). Trace small gravel, sand v. fine.	TOP OF LOESS	
	2-SS	SS	1.5'	3-6-7			Lean CLAY (CL), moist. Dark grey (10YR, 4/1), Light brown 9.4-9.6'. With trace sand and small gravel (few to 1/2").		
10.0									
	3-SS	SS	0.7'	41-50\6"			Silty SAND (SM), moist. Grey (10YR, 5/1). Sand v. fine, uniform. 1" seam of lean clay at 14.0-14.1' bgs w/ some sand & small gravel.	TOP OF WISCONSINAN TILL	
	4-SS	SS	0.8'	27-50\6"			Silty SAND (SM), as above. Lean Clay layer at 18.9-19.2'.		
20.0									
	5-SS	SS	1.0'	22-17-23			Lean CLAY (CL), sl. moist. Dark Grey (10YR, 3/2). With some sand and small gravel. Sand seam (fine) at 23.6-23.8'.		
	6-SS	SS	1.2'	7-12-17			Lean CLAY (CL), as above. Slightly more plastic.		
30.0									

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-2

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint

ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	7-ST	ST	1.7'	NA			Bottom of Tube: Lean CLAY (CL), as above.	Pock. Pen: 2.0, 2.0 TSF. Torvane: 8 TSF	
	8-SS	SS	1.5'	10-20-20			Lean CLAY (CL), as above. Less small gravel (trace).		
	9-SS	SS	1.5'	6-19-22			As Above.		
40.0									
	10-SS	SS	1.5'	5-5-8			Lean CLAY (CL), moist. Very dark grey (10YR, 3/1), with some 1/8" thick black layers w/ slight organic odor. Trace sand & small gravel.	TOP OF INTERGLACIAL ZONE	
	11-SS	SS	1.5'	9-15-21			Lean CLAY (CL), sl. moist. Dark greenish grey (GLEY1, 4/5G), w/ grey mottles. With some sand and trace small gravel.		
50.0									
	12-SS	SS	0.9'	22-50\5"			SILT (ML), moist. Greenish grey (Gley1, 5/5GY). With some sand and small gravel. Slightly plastic.		
	13-SS	SS	1.5'	41-14-16			SILT (ML), moist. Dark grey (10YR, 4/1). With some sand.	TOP OF ILLINOIAN TILL	
60.0									

PROJECT NUMBER: 171881.S1.02.01

BORING NUMBER: B-2

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation

LOCATION: B-2; West Side of ESP Footprint

ELEVATION: 737.8 Feet MSL

DRILLING CONTRACTOR: TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: 3.7' bgs on 8/28/2002

START: 8/2/2002

FINISH: 8/7/2002

LOGGER: MDG

DEPTH BELOW (ft)	SAMPLE: NUMBER	SAMPLE: TYPE	SAMPLE: RECOVERY (FT)	SAMPLE: SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	14-SS	SS	0.8'	32-50\5"	Sandy SILT (ML) and Silty SAND (SM), interbedded in 1" layers, moist to wet. Grey (10YR, 5/1). Slightly plastic.		
70.0	15-SS	SS	1.1'	19-25-28	Interbedded SILT and SAND, as above. Dark grey (10YR, 4/1).		
	16-SS	SS	1.5'	15-23-25	Organic SILT (OH), sl. moist. V. dark greyish brown (10YR, 3/2). V. dilatent, fibrous, some roots. Slight organic odor. Lean CLAY (CL), moist. Grey (10YR, 4/1). With some sand and small gravel. Bottom 0.3': Silty SAND (SW), wet, grey, coarse to fine.		
80.0	17-SS	SS	1.5'	15-27-40	SILT (ML), moist. Dark grey (10YR, 4/1). With some sand and small gravel, slightly plastic.		
	18-SS	SS	0.9'	22-50\5"	SILT (ML), as above.		
90.0	19-SS	SS	0.4'	100\4"	SILT (ML), as above.		

PROJECT NUMBER: 171881.S1.02.01	**BORING NUMBER:** B-2
	Sheet: 4 of 10

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint

ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	20-PIT	PIT	2.7'	NA			End of Tube: Sandy SILT (ML), sl. moist. Grey (10YR, 5/1). With some sand and gravel.	Pock. Pen: >4, >4 TSF. Torvane: > 10 TSF	GR
	21-SS	SS	0.4'	100\3"			Sandy SILT (ML), as above.		
	22-SS	SS	0.2'	100\2"			Sandy SILT (ML), as above.		
100.0									
	23-SS	SS	0.5'	100\6"			Sandy SILT (ML), as above. Dark grey (10YR, 4/1).		
110.0									
	24-SS	SS	0.7'	50-50\3"			Sandy SILT (ML), as above.		
120.0									

REV4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint
ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud
WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	25-SS	SS	1.2'	33-40-50\2			Sandy SILT (ML), as above.		
130.0									
	26-SS	SS	0.9'	38-50\4"			Sandy SILT (ML), as above.		
140.0									
	27-PIT	PIT	2.1'	NA			End of Tube: Sandy SILT (ML), as above.	Pock. Pen: >4, >4 TSF. Torvane: > 10 TSF	GR, C, UU
	28-SS	SS	0.9	66-50\3"			SILT (ML), as above.		
150.0									

PROJECT NUMBER: 171881.S1.02.01	BORING NUMBER: B-2
	Sheet: 6 of 10

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint

ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
160.0							
	29-SS	SS	1.5'	21-38-42	Lean CLAY (CL), moist. Dark olive grey (5Y, 3/2). With some sand and gravel.	TOP OF LACUSTRINE DEPOSIT	
170.0							
	30-SS	SS	0'	25-31-33	No recovery.		
180.0							

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-2

Sheet: 7 of 10

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint

ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	31-PIT	PIT	0'	NA			No recovery.		
								TOP OF PRE-ILLINOIAN TILL	
190.0									GR
	32-PIT	PIT	2.3'	NA			End of Tube: SILT (ML), sl. moist. Dark greyish brown (10YR, 4/2). With some small sand and gravel, few gravel to 1" dia.	Pock. Pen: >4, >4 TSF. Torvane: > 10 TSF	
	33-SS	SS	0.6'	71-50\3"			SILT (ML), as above.		
200.0									
	34-SS	SS	1.3'	33-44-46			Lean CLAY (CL), moist. Dark greyish brown (10YR, 4/2). Trace small gravel and sand.		
210.0									GR

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-2

Sheet: 8 of 10

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint

ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	35-PIT	PIT	2.3'	NA	Lean CLAY (CL), as above.	Pock. Pen: >4, >4 TSF. Torvane: > 10 TSF	
220.0							
	36-SS	SS	0.8'	38-50\3"	Lean CLAY (CL), moist. Dark yellowish brown (10YR, 4/4). Less plastic than above.		
230.0							
	37-SS	SS	0.5'	100\6"	Sandy SILT (ML), sl. moist. Dark greyish brown (10YR, 4/2). Grades with silty fine SAND (SM).		
240.0							

REV4

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-2

Sheet: 9 of 10

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint

ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	38-PIT	PIT	1.9'	NA	Sandy SILT (ML), as above.	Pock. Pen: >4, >4 TSF. Torvane: > 10 TSF	
250.0							
	39-SS	SS	0.8'	41-50\4"	Lean CLAY (CL), moist. Dark greenish grey (GLEY1, 4/10Y). With some sand and small gravel.		
260.0							
	40-PIT	PIT	2.7'	NA	Top of Tube: Lean CLAY (CL), as above. End of Tube: Sandy SILT (ML), moist. Dark greenish grey (GLEY1, 4/10Y). With some sand and fine gravel.	Pock. Pen: >4, >4 TSF. Torvane: > 10 TSF TOP OF PRE-ILLINOIAN ALLUVIUM	GR
	41-SS	SS	1.0'	22-31-38	Fine SAND (SP), moist. Dark greenish grey (GLEY1, 4/10Y). Silty top 0.5'. Sand uniform.		
270.0							

REV4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint
ELEVATION: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud
WATER LEVELS: 3.7' bgs on 8/28/2002 **START:** 8/2/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (FT)	SAMPLE SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
280.0	42-SS	SS	0.5'	100\5.5"			Lean CLAY (CL), sl. moist. Greenish grey (GLEY1, 5/5G) with multiple other colored mottles (reddish brown, grey, tan). Non-horizontal bedding planes present.		GR
290.0									
	43-SS	SS	0.3'	100\2"			Weathered SHALE, dry to sl. moist. Grey.	End of boring at 292.3' bgs. Begin rock coring at 292.3' bgs.	
							End of Boring	Boring abandoned with bentonite slurry to 10' bgs, and bentonite chips to ground surface, upon completion of rock coring and suspension logging.	
300.0									

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-2

Sheet: 1 of 3

ROCK CORE LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint

ELEVATION: Ground: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 3-inch O.D. diamond tip double tube core barrel

WATER LEVEL: NA **START:** 8/7/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	CORE NUMBER	CORE RECOVERY	DISCONTINUITIES: RQD (%)	DISCONTINUITIES: FRACTURES	DISCONTINUITIES: DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS	GRAPHIC LOG	LITHOLOGY: ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS	SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING, ROD DROPS, TEST RESULTS, ETC.
				1	Slightly weathered top 0.3'		SHALE. Dark greenish gray (GLEY2, 4/5BG), with horizontal bedding. Micaceous. Excellent quality, soft. Alternating bands of grey (2.5Y, 5/1).	
1				1				
2				1				
3				1				
4				1				
5	C-1	8.5'	83	2				
6				1	(5.9 - 7.6'): Undulatory irregular bedding, slightly weathered. some fine cross-bedding of sand in 1 mm thin layers within shale.			
7				1				
8				2				Top of coring starts at 292.0' bgs (depth 0.0 on log)
9								
10								

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-2
Sheet: 2 of 3

ROCK CORE LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint
ELEVATION: Ground: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 3-inch O.D. diamond tip double tube core barrel
WATER LEVEL: NA **START:** 8/7/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	CORE NUMBER	CORE RECOVERY	DISCONTINUITIES: RQD (%)	DISCONTINUITIES: FRACTURES	DISCONTINUITIES: DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS	GRAPHIC LOG	LITHOLOGY: ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS	SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING, ROD DROPS, TEST RESULTS, ETC.
				0			SHALE: As above. Abundant thin fine sand and silt layers (<1mm thin)	
11				0				
12				1	(11.7 - 12.2'): Undulatory irregular bedding of fine sand and silt in shale, no predominant orientation			
13				0				
14				2				
15	C-2	10.0'	100	1				
16				0				
17				0				
18				0				
19				0				
20								

ROCK CORE LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-2; West Side of ESP Footprint
ELEVATION: Ground: 737.8 Feet MSL **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 3-inch O.D. diamond tip double tube core barrel
WATER LEVEL: NA **START:** 8/7/2002 **FINISH:** 8/7/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	CORE NUMBER	CORE RECOVERY	DISCONTINUITIES: RQD (%)	DISCONTINUITIES: FRACTURES	DISCONTINUITIES: DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS	GRAPHIC LOG	LITHOLOGY: ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS	SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING, ROD DROPS, TEST RESULTS, ETC.
				0			SHALE: As above. Increasing gray (2.5Y, 5.5/1) interbedding of fine sand and silt.	
21								
				2				
22								
				1				
23								
				1				
24								
				2			Interbedded LIMESTONE and SHALE: Light gray (2.5Y, 7/1). Fossiliferous. Undulatory bedding. Trace pyrite along partings in shale matrix.	
25	C-3	10.0'	79					
				3			COAL: Black. Numerous horizontal and vertical fractures.	
26					Undulatory irregular bedding, vuggy.			
				6				
27							Weathered SHALE: Gray (2.5Y, 5/1). Possibly claystone. Highly weathered throughout.	
				1				
28								End of coring at 30.0' below top of rock.
				2				
29								
				2	(29.3 - 30.0'): Brecchiated with silt infilling, calcarous cement, poss. some limestone fragments.		Brecchiated SHALE: Light gray (2.5Y, 7/1).	
30								

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-3

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint

ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud

WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (FT)	SAMPLE SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
0.0					Ground Surface		
	1-SS	SS	1.0'	5-7-9	Lean CLAY (CL), moist. Olive brown (2.5Y, 3/2). Some fine sand at 4.3-5.0'	TOP OF LOESS	GR
	2-ST	ST	2.0'	NA	Bottom of Tube: As above.	Pock. Pen: 2.0, 2.0 TSF, Torvane: 2.0 TSF	
	3-SS	SS	1.1'	5-6-8	Lean CLAY (CL), moist. Pale brown (10YR, 6/3). With some sand and small gravel.	TOP OF WISCONSINAN TILL	
10.0	4-ST	ST	0'	NA	No recovery.		
	5-SS	SS	1.5'	9-15-16	Lean CLAY (CL), moist. Dark grey (10YR, 4/1). With some small gravel . Becomes SILT (ML) with some fine sand and small gravel, sl. moist.		
	6-ST	ST	1.5'	NA	Bottom of Tube: Lean CLAY (CL), as above.	Pock. Pen: >4, >4 TSF, Torvane: 7 TSF	
	7-SS	SS	1.1'	10-12-16	Lean CLAY (CL), as above. Gravel to 1/2" dia.		
20.0							
	8-SS	SS	1.1'	5-7-9	Lean CLAY (CL), more plastic than above.		
	9-SS	SS	1.4'	5-6-7	Lean CLAY (CL), moist. Dark grey (10YR, 4/1). With some sand and small gravel.		
30.0							GR

REV4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint

ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud

WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	10-ST	ST	1.7'	NA	Lean CLAY (CL), as above.	Pock. Pen: 1.5, 1.25 TSF, Torvane: 5 TSF	
	11-SS	SS	1.7'	10-15-17	As above.		
	12-SS	SS	0.3'	6-6-6	Organic SILT (ML), sl. moist. V. dark brown (10YR, 2/2). Some fibers, slight organic odor. Angular gravel fragment (3/4") at 39.9' bgs.	TOP OF INTERGLACIAL ZONE	
40.0	13-ST	ST	2.0'	NA	Bottom of Tube: Lean CLAY (CL), moist. Dark grey (10YR, 4/1). With some sand.	Pock. Pen: 1.25, 1.25 TSF, Torvane: 4.5 TSF	
	14-SS	SS	0.7'	6-8-11	Sandy CLAY (CL), moist. Grey (GLEY1, 5/N). Sand uniform.		
	15-ST	ST	2.0'	NA	Bottom of Tube: Silty SAND (SM), moist. Grey (GLEY1, 5/N). Very fine & uniform.	Pock. Pen: 1.25, 1.75 TSF, Torvane: 4.0 TSF	
	16-SS	SS	0.7'	16-21-35	Silty SAND (SW), wet. Grey (10YR, 5/1).		
50.0							
	17-SS	SS	0.9'	21-50\4"	Sandy SILT (ML), moist. Grey (10YR, 5/1). With some gravel to 1/4".		
	18-PIT	PIT	2.3'	NA	Bottom of Tube: as above.	Pock. Pen: >4, >4 TSF, Torvane: >10 TSF	GR
	19-SS	SS	1.4'	44-50\5"	Sand (SW), wet. Grey (10YR, 5/1). Fine to coarse, rounded. Becomes sandy SILT (SM) bottom 0.7'.	TOP OF ILLINOIAN TILL	
60.0							

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-3

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint

ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud

WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	20-PIT	PIT	1.5'	NA			Bottom of Tube: Sandy SILT (ML), sl. moist. Grey (10YR, 5/1). With some sand and small gravel (rounded, to 3/4" dia.).	Pock. Pen: >4, >4 TSF, Torvane: 5.5 TSF	
	21-SS	SS	0.3'	100\4"			As above.		
	22-SS	SS	1.4'	12-19-25			Sandy SILT (ML), moist. Greenish grey (GLEY1, 5/10R). With some sand and small gravel. Slightly plastic.		
70.0									
	23-PIT	PIT	2.4'	NA			As above, less plastic.	Pock. Pen: >4, >4 TSF, Torvane: 6.5 TSF	
	24-SS	SS	1.1'	21-38-42			As above.		
	25-SS	SS	0.7'	25-36-50\2			Sandy SILT (ML), sl. moist. Dark grey (10YR, 4/1). With sand and small gravel.		
80.0									
	26-PIT	PIT	2.1'	NA			Bottom of Tube: as above.	Pock. Pen: >4, >4 TSF, Torvane: >10 TSF	GR
	27-SS	SS	0.5'	100\5.5"			As above. Dark grey (10YR, 4/1).		
	28-SS	SS	0.4'	100\5"			As above. 1" dia. gravel fragment at 88.8' bgs.		
90.0									

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint
ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud
WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	29-SS	SS	0.3'	100\3"	As above.		
	30-PIT	PIT	2.8'	NA	Bottom of Tube: as above.	Pock. Pen: >4, >4 TSF, Torvane: >10 TSF	
	31-SS	SS	0.4'	100\5"	As above.		
100.0							
	32-SS	SS	0.3'	100\4"	Sandy SILT (ML), as above. Slightly more plastic.		
110.0							
	33-PIT	PIT	2.7'	NA	Bottom of Tube: as above. Dry to sl. moist.	Pock. Pen: >4, >4 TSF, Torvane: >10 TSF	
	34-SS	SS	0.8'	50-50\3"	As above.		
120.0							

PROJECT NUMBER: 171881.S1.02.01 BORING NUMBER: B-3

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint

ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud

WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	35-SS	SS	0.8'	61-50\3"			As above.		
130.0									
	36-PIT	PIT	2.7'	NA			Bottom of Tube: as above.	Pock. Pen: >4, >4 TSF, Torvane: >10 TSF	GR
	37-SS	SS	0.8'	62-50\4"			As above.		
140.0									
	38-SS	SS	0.5'	75\6"			As above.		
150.0									

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-3

Sheet: 6 of 10

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint

ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud

WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	39-PIT	PIT	2.7'	NA	Bottom of Tube: Sandy SILT (ML), as above.	Pock. Pen: >4, >4 TSF, Torvane: 8 TSF	
	40-SS	SS	0.9'	47-50\3"	As above.		
160.0							
	41-SS	SS	0.8'	48-50\2"	Sandy CLAY (CL), moist. Olive (5Y, 4/3), with some grey (10YR, 5/1) mottling. With some sand and small gravel.	TOP OF LACUSTRINE DEPOSITS	
170.0	42-PIT	PIT	2.7'	NA	Top of Tube: as above. Bottom of Tube: as above, dark greenish grey (GLEY1, 4/5GY)	Pock. Pen: >4, >4 TSF, Torvane: >10 TSF	GR, C, CU
	43-SS	SS	0.5'	100\6"	As above, dark greenish grey (GLEY1, 4/10Y)		
180.0							

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint
ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud
WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE: NUMBER	SAMPLE: TYPE	SAMPLE: RECOVERY (FT)	SAMPLE: SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	COMMENTS: TEST DATA
	44-SS	SS	0.8'	49-50\3"	Sandy SILT (ML), sl. moist. V. dark grayish brown (10YR, 3/2). With sand and small gravel.	TOP OF PRE-ILLINOIAN TILL	
190.0							
200.0	45-PIT	PIT	0'	NA	No recovery.		
	46-SS	SS	0.8'	50-50\3"	Bottom of tube: SILT (ML), dry to sl. moist. Dark grey (10YR, 4/1). Trace fine sand. Some horizontal bedding.		
	47-PIT	PIT	2.8'	NA	Bottom of tube: as above. Slightly more plastic.	Pock. Pen: 3.5, 3.75 TSF, Torvane: 6 TSF	
210.0	48-SS	SS	0.7'	47-50\2"	Top 3": as above. Bottom 4": Fine silty SAND (SP), wet. Grey (10YR, 5/1). Uniform.		

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-3

Sheet: 8 of 10

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint

ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud

WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	49-SS	SS	0.5'	50\6"			Silty <u>SAND (SW)</u>, wet. Grey (10YR, 5/1).		
220.0									
	50-PIT	PIT	1.7'	NA			Bottom of tube: <u>Lean CLAY (CL)</u>, moist. Olive grey (5Y, 4/2). Trace sand and small gravel.	Pock. Pen: >4, >4 TSF, Torvane: >10 TSF	GR
	51-SS	SS	1.4'	51-50\4"			As above. Some gravel to 3/4". Some gray mottles. Grades to dark grayish brown (10YR, 4.5/2).		
230.0									
	52-SS	SS	1.3'	41-50\5"			As above. Dark grayish brown.		
240.0									

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-3

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint

ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud

WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	53-PIT	PIT	2.8'	NA	Bottom of tube: as above.	Pock. Pen: >4, >4 TSF, Torvane: >10 TSF	
	54-SS	SS	0.8'	NR	Top 4": as above. Bottom 5": SILT (ML), wet. Grayish brown (10YR, 5/2). Highly dilatent, non-plastic.	TOP OF PRE-ILLINOIAN ALLUVIUM	
250.0							
	55-SS	SS	1.5'	17-28-33	Lean CLAY (CL), moist. V. dark greenish grey (GLEY2, 4/5BG). With sand and trace small gravel.		
260.0							
	56-SS	SS	0.8'	3-17-30\3	SILT (ML), moist. Dark gray (10YR, 4/1). Horizontal striations/bedding, possibly fibers.		
270.0							

REV4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint

ELEVATION: 734.2 **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6" Rotary, Bentonite Drill Mud

WATER LEVELS: 5.5' bgs on 8/28/2002 **START:** 7/26/02 **FINISH:** 8/1/02 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (FT)	SAMPLE SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	57-SS	SS	1.4'	22-50\5.5"			Top 3": Gravelly and sandy CLAY (CH), moist. Clay brown (10YR, 5/3) with dark grey mottles. Gravel to 3/4" dia., angular and rounded. May be > 50% gravel. Bottom 1.1': Fat CLAY (CH), moist. Dark greenish grey (GLEY1, 4/10Y) with bluish gray mottles (GLEY2, 6/1). Little gravel to 1/2" dia. Weathered sandstone inclulsion at 1.1'.		
280.0									
							End of Boring	Hard drilling (with bit chatter) began at 284'. Continued hard drilling to 286'. Split spoon attempted at 286' bounced. Assumed top weathered bedrock at 284'. Begin rock coring at 286.2'.	
290.0								Boring abandoned with bentonite slurry to 10' bgs, and bentonite chips to ground surface, upon completion of rock coring.	
300.0									

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-3
Sheet: 1 of 2

ROCK CORE LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint
ELEVATION: Ground: 734.2 Feet MSL **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 3-inch O.D. diamond tip double tube core barrel
WATER LEVEL: NA **START:** 8/1/2002 **FINISH:** 8/1/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	CORE NUMBER	CORE RECOVERY	DISCONTINUITIES: RQD (%)	DISCONTINUITIES: FRACTURES	DISCONTINUITIES: DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS	GRAPHIC LOG	LITHOLOGY: ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS	SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING, ROD DROPS, TEST RESULTS, ETC.
				3			SANDSTONE. Gray (2.5Y, 5.5/1). Calcarous cementation, massive. Fine grained, uniform, silty.	
1							SHALE. Gray (2.5Y, 5/1). Micaceous, with planar beddings. Abundant fine sand to silt partings.	
				1	Weathered zone, some clay infilling at 1.6-1.7' (bluish grey)			
2								
				2	Weathered zones at 2.2-2.3' and 2.7-2.8'			
3								
				0				
4								
				1				
5	C-1	10.0'	93 %					
				1	Grades to dark gray (2.5Y, 4/1) 5.0 to 6.0'			
6								
				0				
7					Irregular undulatory silty fine sand bedding.			
				1				
8								Top of coring starts at 286.2' bgs (depth 0.0 on log)
				0				
9								
				0				
10								

ROCK CORE LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-3; Eastern side of ESP footprint
ELEVATION: Ground: 734.2 Feet MSL **DRILLING CONTRACTOR:** TSC
DRILLING METHOD AND EQUIPMENT: 3-inch O.D. diamond tip double tube core barrel
WATER LEVEL: NA **START:** 8/1/2002 **FINISH:** 8/1/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	CORE NUMBER	CORE RECOVERY	DISCONTINUITIES: RQD (%)	DISCONTINUITIES: FRACTURES	DISCONTINUITIES: DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS	GRAPHIC LOG	LITHOLOGY: ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS	SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING, ROD DROPS, TEST RESULTS, ETC.
							SHALE. As above. Fewer silt and fine sand partings (<1mm thin).	
				2	Weathered zone, some clay infilling at 10.5-10.7'. Vertical fracture 10.4 to 11.4'.			
11								
				1				
12								
				1				
13								
				0				
14								
				0				
15	C-2	10.0'	96 %					
				0				
16								
				0				
17								
				0				
18								End of rock coring at 20.0' below top of rock
				0				
19								
				0				
20								

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-4

Sheet: 1 of 4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-4; Southeast corner of ESP footprint

ELEVATION: 735.4 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: NA **START:** 7/24/2002 **FINISH:** 7/25/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
0.0					Ground Surface		
	1-SS	SS	1.5'	7-5-8	Silt (ML) and Clay (CL), moist. Varies grey (10Y, 4/1) and light olive brown (2.5YR, 5/3). With some small gravel and sand seams. Black (10YR, 2/1) with roots bottom 0.5'.	TOP OF LOESS	
	2-ST	ST	2.0'	NA	Bottom of Tube: As above (4.5'-5.0'), no roots	Pock. Pen: 2.0, 2.0 TSF Torvane: 8.0 TSF	
	3-SS	SS	0.8'	3-4-6	Lean CLAY (CL), moist. Black (10YR, 2/1) with grey (10YR, 5/1) and yellowish brown (10YR, 5/8) mottles. Trace fine sand.	TOP OF WISCONSINAN TILL	
10.0	4-ST	ST	1.3'	NA	Bottom of Tube: As above.	Pock. Pen: 0.75, 0.75 TSF Torvane: 3.0 TSF	
	5-SS	SS	1.5'	4-4-5	Lean CLAY (CL), moist. Brown (10YR, 5/3). With some small gravel.		
	6-ST	ST	0.4'	NA	Bottom of tube: As above.	Pock. Pen: 1.5, 1.5 TSF Torvane: 3.0 TSF	
	7-SS	SS	1.5'	4-5-8	Lean CLAY (CL), moist. Very dark grey (10YR, 3/1). Trace small gravel.		
20.0	8-ST	ST	0'	NA	No recovery.		
	9-SS	SS	0'	26-31-49	No recovery.		
	10-PIT	PIT	2.2'	NA	Bottom of tube: As above (18.5'-20.0')	Pock. Pen: 3.75, 4.0 TSF Torvane: 6.5 TSF	
30.0	11-SS	SS	0.9'	8-10-11	As above.		

PROJECT NUMBER: 171881.S1.02.01 BORING NUMBER: B-4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-4; Southeast corner of ESP footprint

ELEVATION: 735.4 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: NA **START:** 7/24/2002 **FINISH:** 7/25/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	12-SS	SS	1.5'	6-8-8	As above. Some gravel to 1" dia.		
	13-ST	ST	0.9'	NA	Bottom of tube: As above. 1" dia. gravel (2 pieces) in bottom.	Pock. Pen: 1.0, 1.0 TSF Torvane: 4.0 TSF	
	14-SS	SS	1.5'	8-11-16	Organic SILT (ML), slightly moist. Dark greyish brown (10YR, 4/2), grades to v. dark greyish brown. Fibrous (roots?), slight organge.	TOP OF INTERGLACIAL ZONE	
40.0	15-ST	ST	0.4'	NA	Bottom of tube: As above.	Pock. Pen: 1.0, 1.0 TSF Torvane: 3.5 TSF	GR
	16-ST	SS	1.5'	7-8-11	Lean CLAY (CL), moist. Grey and dark grey mottles (10YR, 5/1 and 4/1). With some small gravel and silt.		
	17-ST	ST	0'	NA	No recovery.		
	18-SS	SS	1.5'	8-8-12	(48.5' - 50.0'): Clayey SAND (SC), and Sandy CLAY (CL), mois		
50.0	19-ST	ST	2.0'	NA	Top of Tube: As above. Bottom of Tube: Clayey SAND (SC), moist. Grey (10YR, 5/1). Well graded, angular.	Pock Pen & Torvane: NA (sand)	GR
	20-SS	SS	0.9'	25-26-28	Clayey SAND (SC), as above.		
						TOP OF ILLINOIAN TILL	
	21-SS	SS	0.8'	10-12-16	Sandy SILT (ML), moist. Grey (10YR, 5/1). With some well graded gravel, to 3/4" dia.		
60.0							

PROJECT NUMBER: 171881.S1.02.01 **BORING NUMBER:** B-4

Sheet: 3 of 4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-4; Southeast corner of ESP footprint

ELEVATION: 735.4 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: NA **START:** 7/24/2002 **FINISH:** 7/25/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (FT)	SPT-N (6"-6"-6")	VISUAL LOG	SAMPLE SYM.	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	22-PIT	PIT	1.9'	NA			Bottom of tube: Sandy SILT (ML), moist. Grey (10YR, 5/1). With some sand and small gravel.)	Pock. Pen: >4, >4 TSF Torvane: 7.0 TSF	GR
	23-SS	SS	1.3'	12-15-17			As above.		
	24-SS	SS	1.4'	11-16-17			As above.		
70.0									
	25-SS	SS	1.0'	16-21-23			As above.		
	26-PIT	PIT	0'	NA			Bottom of tube: Large cobble (3" dia.) obstructed end of tube. Cobble rounded, basalt.	Pock. Pen & Torvane: NA (no recovery)	
	27-SS	SS	0.8'	47-50\4"			As above Sandy SILT (ML).		
80.0									
	28-PIT	PIT	2.0'	NA			Bottom of tube: As above.	Pock. Pen: >4, >4 TSF Torvane: >10 TSF	
	29-SS	SS	0.3'	100\4"			As above.		
	30-SS	SS	0.7'	44-50\3"			As above.		
90.0									

PROJECT NUMBER: 171881.S1.02.01	BORING NUMBER: B-4
	Sheet: 4 of 4

SOIL BORING LOG

PROJECT: Exelon - CPS-ESP Field Investigation **LOCATION:** B-4; Southeast corner of ESP footprint

ELEVATION: 735.4 Feet MSL **DRILLING CONTRACTOR:** TSC

DRILLING METHOD AND EQUIPMENT: 6-Inch Rotary, Bentonite Drilling Mud

WATER LEVELS: NA **START:** 7/24/2002 **FINISH:** 7/25/2002 **LOGGER:** MDG

DEPTH BELOW (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (FT)	SAMPLE SPT-N (6"-6"-6")	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION	TEST DATA
	31-SS	SS	0.3'	100\3"	Sandy SILT (ML), as above.		
	32-PIT	PIT	1.7'	NA	Bottom of tube: As above Sandy SILT (ML), 1 1/2" dia. cobble (quartzite, rounded) in bottom.	End of Boring at 100.0' bgs. Abandoned with 125 gallons of bentonite grout	GR
	33-SS	SS	0.3'	100\4"	As above Sandy SILT (ML).		
100.0					End of Boring		
110.0							
120.0							

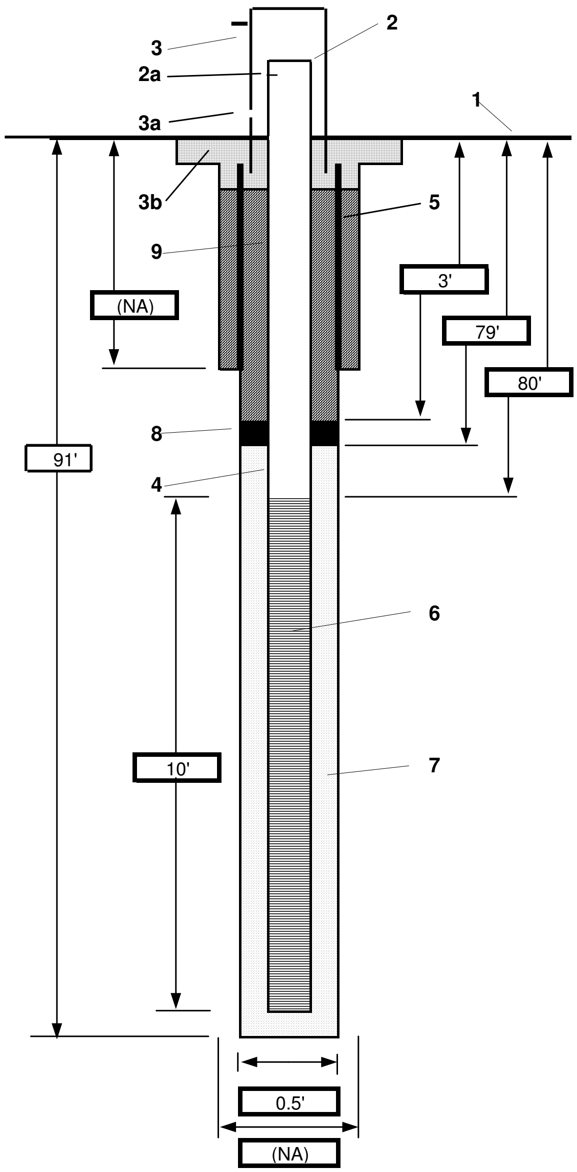
Attachment A-2

Piezometer Construction Logs

PROJECT NUMBER	WELL NUMBER	
171881.S1.02.01	**B-1 Piezo**	SHEET 1 OF 1

WELL COMPLETION DIAGRAM

PROJECT : ESP Geotechnical Investigation **LOCATION :** ESP Footprint, SW Corner
DRILLING CONTRACTOR : Testing Service Corporation (TSC)
DRILLING METHOD AND EQUIPMENT USED : 6" Rotary Drilling, with BioBore Drilling Mud
WATER LEVELS : 29.8' BTOC on 8/6/02 **START :** 7/23/02 **END :** 07/23/02 **LOGGER :** MDG

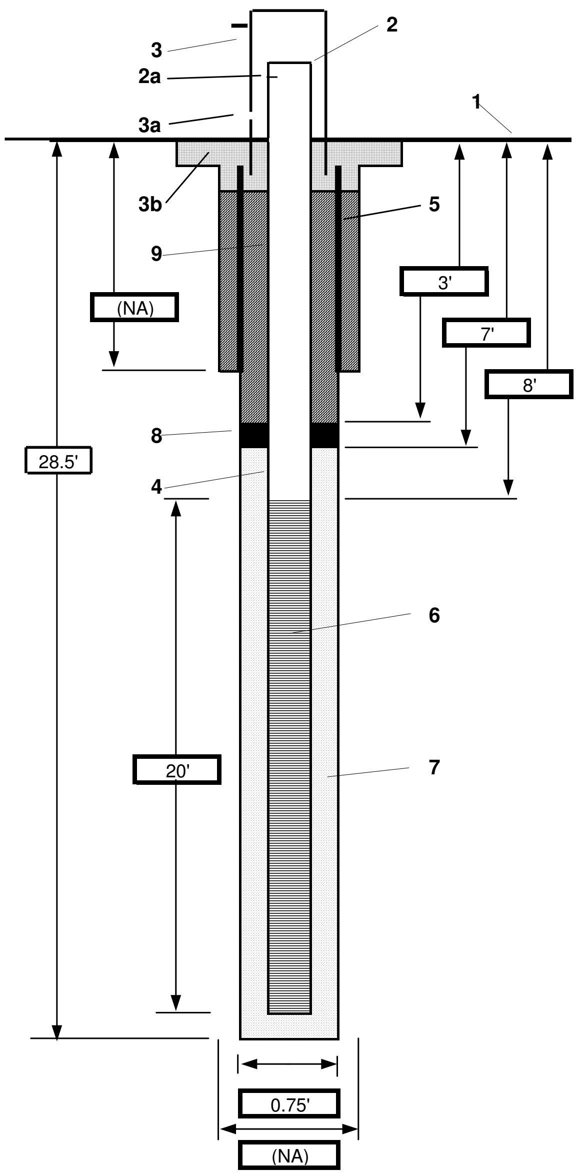


1- Ground elevation at well	738.59 feet above MSL (top concrete)
2- Top of casing elevation	740.92 feet above MSL
a) vent hole?	Vent hole in cap
3- Wellhead protection cover type	4-inch dia. steel casing, round
a) weep hole?	No
b) concrete pad dimensions	2-feet square, formed
4- Dia./type of well casing	2-inch dia. Sch. 40 PVC
5- Dia./type of surface casing	No surface casing left in place
6- Type/slot size of screen	2-inch dia. PVC. 0.010" milled slots
7- Type screen filter	# 5 silica sand
a) Quantity used	4 bags @ 50 pounds/bag
8- Type of seal	Hydrated "Hole Plug" bentonite chips
a) Quantity used	1 bag (50 pounds)
9- Grout	
a) Grout mix used	No grout - hydrated "Hole Plug" used
b) Method of placement	Slow placement from top of borehole
c) Vol. of surface casing grout	NA
d) Vol. of well casing grout	NA (19 bags @ 50 pounds/bag)
Development method	Purge 1 piezometer volume with bailer
Development time	NA
Estimated purge volume	18.9 gallons
Comments 1)	Water level dropped 16.5' by the end of purging at B-1
2)	Bottom 9 feet of borehole (to 100' bgs) was filled with bentonite chips prior to installing the piezometer

CH2MHILL	PROJECT NUMBER 171881.S1.02.01	WELL NUMBER **B-2 Piezo**	SHEET 1 OF 1

WELL COMPLETION DIAGRAM

PROJECT : ESP Geotechnical Investigation **LOCATION :** ESP Footprint, Western Half
DRILLING CONTRACTOR : Testing Service Corporation (TSC)
DRILLING METHOD AND EQUIPMENT USED : 3 3/4" ID Hollow-Stem Augers
WATER LEVELS : 8.99' on 8/6/02 **START :** 7/25/02 **END :** 07/25/02 **LOGGER :** MDG



1- Ground elevation at well 737.17 feet above MSL

2- Top of casing elevation 739.55 feet above MSL
a) vent hole? Vent hole in cap

3- Wellhead protection cover type 4-inch dia. steel casing, round
a) weep hole? No
b) concrete pad dimensions 2-feet square, formed

4- Dia./type of well casing 2-inch dia. Sch. 40 PVC

5- Dia./type of surface casing No surface casing left in place

6- Type/slot size of screen 2-inch dia. PVC. 0.010" milled slots

7- Type screen filter # 5 silica sand
a) Quantity used 9 bags @ 50 pounds/bag

8- Type of seal Hydrated "Hole Plug" bentonite chips
a) Quantity used 1 bag (50 pounds)

9- Grout
a) Grout mix used No grout - hydrated "Hole Plug" used
b) Method of placement Slow placement from top of borehole
c) Vol. of surface casing grout NA
d) Vol. of well casing grout NA (1 bag @ 50 pounds/bag)

Development method Purge 1 piezometer volume with bailer

Development time NA

Estimated purge volume 19.1 gallons

Comments Water level dropped 19.5' by the end of purging at B-2

CH2MHILL	PROJECT NUMBER 171881.S1.02.01	WELL NUMBER B-3 Piezo	SHEET 1 OF 1

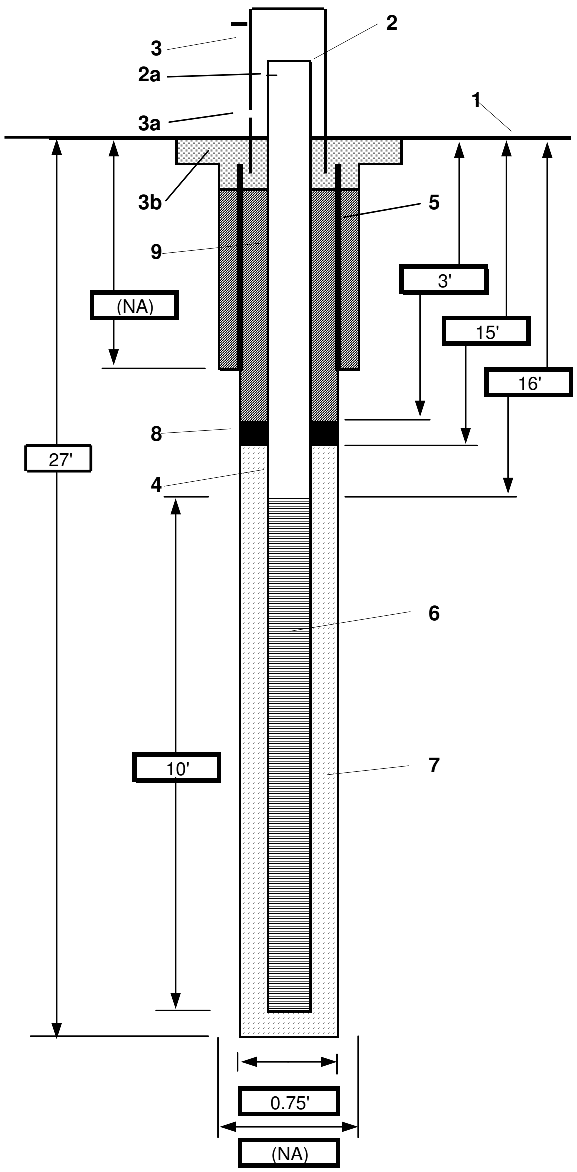
WELL COMPLETION DIAGRAM

PROJECT : ESP Geotechnical Investigation **LOCATION :** ESP Footprint, Eastern Half

DRILLING CONTRACTOR : Testing Service Corporation (TSC)

DRILLING METHOD AND EQUIPMENT USED : 3 3/4" ID Hollow-Stem Augers

WATER LEVELS : 11.64' on 8/6/02 **START :** 7/25/02 **END :** 07/25/02 **LOGGER :** MDG



1- Ground elevation at well	734.06 feet above MSL (top concrete)
2- Top of casing elevation	736.37 feet above MSL
a) vent hole?	Vent hole in cap
3- Wellhead protection cover type	4-inch dia. steel casing, round
a) weep hole?	No
b) concrete pad dimensions	2-feet square, formed
4- Dia./type of well casing	2-inch dia. Sch. 40 PVC
5- Dia./type of surface casing	No surface casing left in place
6- Type/slot size of screen	2-inch dia. PVC. 0.010" milled slots
7- Type screen filter	# 5 silica sand
a) Quantity used	6 bags @ 50 pounds/bag
8- Type of seal	Hydrated "Hole Plug" bentonite chips
a) Quantity used	1 bag (50 pounds)
9- Grout	
a) Grout mix used	No grout - hydrated "Hole Plug" used
b) Method of placement	Slow placement from top of borehole
c) Vol. of surface casing grout	NA
d) Vol. of well casing grout	NA (4 bags @ 50 pounds/bag)
Development method	Purge 1 piezometer volume with bailer
Development time	NA
Estimated purge volume	18.0 gallons
Comments	Water level dropped 14.4' by the end of purging at B-3

Attachment A-3

GRL SPT Hammer Calibration Test Report

GRL Engineers, Inc.
Goble Rausche Likins and Associates, Inc.
(Formerly)

August 12, 2002

Mr. Don Anderson
CH2MHill
777 108th Avenue NE
Bellevue, WA 98004

Re: SPT Energy Measurement Summary Report
Clinton Power Station
Clinton, Illinois

GRL Job No. 027049

Dear Mr. Anderson:

This report summarizes the results from the dynamic energy measurements made on one SPT drill rig at the above referenced site on August 2, 2002. A preliminary summary of the test results was previously transmitted following the completion of the field work.

The purpose in making the dynamic measurements was to determine the average energy transfer from Testing Service Corporation's (TSC) manual Standard Penetration Test (SPT) safety hammer to NW-J drill rod during SPT sampling events. A PAK model, Pile Driving Analyzer ® (PDA) acquired and processed the dynamic test data to meet these test objectives. Additional information on the testing equipment, details on the analytical procedures, as well as limitations of dynamic test methods are presented in Exhibit A of this report.

Drill Rig and SPT Hammer Details

Testing was conducted on a truck mounted Gus Pech 7500 drill rig. This drill rig was equipped with a manual SPT safety hammer operated by a cathead and rope system. The drill rod used was NW rod with a J taper thread. The energy measurements were made using a 2 foot long instrumented NW rod segment inserted in the drill string immediately below the anvil of the hammer.

DYNAMIC TEST FIELD DETAILS

Instrumentation

A Pile Driving Analyzer (PDA) was used to process dynamic measurements of strain and acceleration taken on the 2 foot long NW rod segment located between the safety hammer and drill string. The strain and acceleration signals were conditioned and converted to forces and velocities by the PDA. During SPT sampling, the PDA calculated values for the maximum force, the maximum impact velocity, the hammer operating rate, the transferred hammer energy to the gage location, and the energy transfer ratio. Calibration information for the strain gages attached to the instrumented rod segment and accelerometers used is included in Exhibit C.

Force and velocity records from the PDA were also viewed on a graphic LCD screen during sampling to evaluate data quality. Force and velocity records were digitally stored on disk for subsequent laboratory analysis.

Test Sequence

On August 2, 2002 energy measurements were made during SPT sampling in Boring 2 at the Clinton Power Station. The soil boring had been advanced to a depth of 43.5 feet prior to the first instrumented sampling event. Three TSC personnel operated the cathead and rope system as ten split spoon samples were collected at five foot intervals between the depths of 43.5 and 88.5 feet. Six of the ten split-spoon samples were driven 18 inches as blows were recorded for each of the three 6 inch increments. The SPT N value for each sampling event was then calculated as the number of blows for the final 12 inches of driving. Sampler refusal occurred during four of the sampling events. For three of these event Exhibit pler refused in the second 6 inch increment after 50 blows. For the fourth of these events, the sampler refused in the first increment after a reported 100 blows.

DYNAMIC TESTING ANALYSIS DETAILS

Case Method

The PDA interprets the measured dynamic data according to the Case Method equations. The dynamic test data was evaluated for the energy transferred to the gage location, (EFV); the energy transfer ratio, (ETR); the maximum impact force at the gage location, (FMX); the maximum impact velocity at the gage location, (VT1); and the SPT hammer operating rate, (BPM). These quantities are presented in the summaries of the dynamic test results in Exhibit B.

The maximum energy transfer to the gage location was calculated by integrating both the force and velocity records over time as follows:

$$EFV = \int F(t)V(t)dt$$

Where: F = the force at time t
V = the velocity at time t

The integration begins at the hammer impact time and continues until the maximum transferred energy is reached. Using this equation, the average energy transfer over the SPT N value increment was computed and is presented in Table 1. Data from the seating blows in the first 6 inch increment was not used in this calculation.

A summary of the energy measurement results for each cathead and rope operator is presented below:

Operator	Number of Samples	Average Transferred Energy (ft-lbs)	Average Energy Transfer Ratio (%)	Range in Transferred Energy (ft-lbs)	Range in Energy Transfer Ratio (%)
Francisco	4	194	55	166 - 205	47 - 58
Dave	4	167	48	136 - 191	39 - 55
Greg	2	195	56	182 - 208	52 - 60

The maximum, minimum, and standard deviation in energy transfer for each SPT sample over the SPT N value increment are included in Exhibit B.

Table 1 also includes the reported SPT N value and the SPT N value corrected for 60% transferred energy, N_{60}. The N_{60} value was calculated using the Schmertman correction as follows:

$$N_{60} = (e_m / 60)\, N_m$$

Where: e_m = the measured transferred energy ratio
N_m = the measured SPT N value.

For informational purposes, the energy transfer to the drill rod string was also calculated using the EF2 equation. This was the method specified in ASTM D-4633-86, Standard Test Method for Stress Wave Energy Measurement for Dynamic Penetrometer Testing Systems. In this equation, the transfer energy is calculated assuming proportionality exists between force and velocity so that the transferred energy can be calculated in terms of only one measured quantity, the force. By assuming that force and velocity are proportional, the EF2 equation assumes that the drill rod is of constant cross sectional area which is seldom the case. For this reason, we have not presented the EF2 results in Table 1 or recommend their use.

The EF2 equation can be expressed as:

$$EF2 = (c/EA)\int[f(t)]^2\,dt$$

Where: c = the stress wave speed in the drill rod
E = Modulus of Elasticity of the drill rod
A = area of the drill rod at the gage location
F = the force at time t

In the EF2 equation, the integration begins at the hammer impact time and continues to a cutoff time that corresponds to the first occurrence of a zero force after impact. The ASTM standard required that the cutoff time fall within a time of 0.9(2L/c) to 1.2(2L/c), where L is the length between the gage location and the bottom of the sampler. ASTM D-4633 also required that several correction factors be applied based on the distance between the impact point and measuring station, the overall rod length, and a velocity correction factor. The energy calculated from this method is contained in the summary tables presented in Exhibit B. This data complies with the ASTM cutoff times. No other ASTM corrections have been applied. It should be noted that this ASTM standard expired in 1995 and a current ASTM standard does not exist.

CONCLUSIONS

Based upon the dynamic test data obtained, the following conclusions are presented:

1) Ten SPT sampling events were performed with TSC's Gus Pech 7500 drill rig utilizing a safety hammer hoisted by a cathead and rope system. The average energy transfer to the drill rod for an individual sampling event ranged from 136 to 208 ft-lbs. This corresponds to an energy transfer ratio of 39 to 60% of the 350 ft-lbs theoretical SPT hammer energy. The overall average for the eight sampling events was 183 ft-lbs or 52% of the theoretical energy.

August 12, 2002

2) The ten sampling events with the safety hammer/cathead and rope system were performed by three drillers: Francisco, Dave, and Greg. The average energy transfer for the four SPT sampling events performed by Francisco was 194 ft-lbs, and the average energy transfer for the four SPT sampling events performed by Dave was 167 ft-lbs. For the two sampling events performed by Greg, the average energy transfer was 195 ft-lbs.

3) Sampler refusal occurred during four of the ten sampling events. For three of these events, the sampler refused in the second 6 inch increment after 50 blows. For the fourth of these events, the sampler refused in the first increment after a reported 100 blows.

4) Variation between the reported SPT blow counts and the PDA recorded number of blows occurred for some of the samples. For these samples, the variation was accounted for by adjusting the number of "seating blows" in the first 6 inch SPT increment.

We appreciate the opportunity to be of assistance to you on this project. Please do not hesitate to contact us if you have any questions regarding this report, or if we may be of further service.

Sincerely,

GOBLE RAUSCHE LIKINS
AND ASSOCIATES. INC.

Mark A. Rawlings

Mark A. Rawlings

Patrick J. Hannigan

Patrick J. Hannigan, P.E.

TABLE 1: Summary of SPT Hammer Energy Transfer Measurements
Clinton Power Station, Clinton, Illinois

Operator	Sample Number	Sample Starting Depth (ft)	Reported SPT Values Per 6 Inch Increment	SPT Field N Value	Average Energy Transfer To Rod [(1)] (ft-lbs)	Average Energy Transfer Ratio [(2)] (%)	SPT N Value Corrected for 60% Energy [(3)] N_{60}	SPT Hammer Operating Rate (blows / min.)
Francisco	10-SS	43.5	5-5-8	13	199	57	12	50.1
	13-SS	58.5	21-14-16	30	204	58	29	50.3
	15-SS	68.5	19-25-28	53	205	59	52	49.7
	18-SS	83.5	22-50/5"	50/5"	166	47	40/5"	52.8
Dave	11-SS	48.5	9-15-21	36	191	55	33	51.8
	14-SS	63.5	32-50/5"	50/5"	169	48	40/5"	53.0
	16-SS	73.5	15-25-23	48	172	49	39	51.6
	19-SS	88.5	100/4"	100/4"	136	39	65/4"	54.3
Greg	12-SS	53.5	22-50/5"	50/5"	182	52	43/5"	50.6
	17-SS	78.5	15-27-40	67	208	60	67	50.4

Notes:

1) - Average energy transfer over second and third increment from FV Method.
2) - Average energy transferred to drill rod divided by 350 ft-lbs.
3) - SPT N value corrected for 60% energy using the Schmertman Correction Method.

All samples were taken with a split spoon sampler.
A cathead and rope system was utilized to hoist the safety hammer.
The drill rig was a truck mounted Gus Pech 7500.

EXHIBIT A

An Introduction Into
Dynamic Testing Methods

EXHIBIT A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

1. BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during both preconstruction test programs and production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (e.g. a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the "High Strain Test" Method of pile testing. However, for the sake of completeness, two types of "Low Strain Tests" are also mentioned: the Pile Integrity Test™ (PIT) and Cross Hole Sonic Logging conducted with the Cross Hole Analyzer (CHA).

2. RESULTS FROM PDA DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring.*
- *Dynamic Load Testing.*

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both drilled shafts and impact driven piles during restrike.

2.1 DYNAMIC PILE MONITORING

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- *Bearing capacity* at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- *Dynamic pile stresses* axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- *Pile integrity* assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- *Hammer performance* parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

2.2 DYNAMIC PILE LOAD TESTING

Bearing capacity testing of either driven piles or drilled shafts employs the basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it is most important that the test is conducted after a sufficient waiting time following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- *Bearing capacity, i.e.* the mobilized capacity present at the time of testing.

- *Resistance distribution* including shaft resistance and end bearing components.

- *Stresses in pile or shaft* calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.

- *Shaft impedance vs. depth*; this is an estimate of the shaft shape if it differs substantially from the planned profile.

- *Dynamic soil parameters* for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface).

3. MEASUREMENTS

The following is a general summary of dynamic measurements available to solve typical deep foundation problems.

3.1 PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates, and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

3.2 HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

3.3 SAXIMETER™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

3.4 PIT

The Pile Integrity Tester™ (PIT) helps in detecting major defects in concrete piles or shafts or assess the length of a variety of deep foundations, except steel piles. PIT performs the so-called "Pulse-Echo Method" which only requires the measurement of motion (e.g., acceleration) at the pile top caused by a light hammer impact. PIT also supports the so-called "Transient Response Method" which requires the additional measurement of the hammer force and an analysis in the frequency domain. PIT may also be used to evaluate the unknown length of deep foundations under existing structures.

3.5 CHA

This test requires that at least two tubes (typically steel tubes of 50 mm diameter) are installed vertically in the shaft to be tested. A high frequency signal is generated in one of the water filled tubes and received in the other tube. The received signal strength and its First Time of Arrival (FTA) yield important information about the concrete quality between the two tubes. The transmitting and recording of the signal is repeated typically every 50 mm starting at the shaft bottom and all records together establish a log or profile of the concrete quality between the two tubes. The total number of tubes installed depends on the size of the drilled shaft. The more tubes are present the more profiles can be constructed.

4.ANALYTICAL SOLUTIONS

4.1 *BEARING CAPACITY*

4.1.1 WAVE EQUATION

GRL has written the GRLWEAP™ program which calculates a relationship between bearing capacity, pile stress, and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph

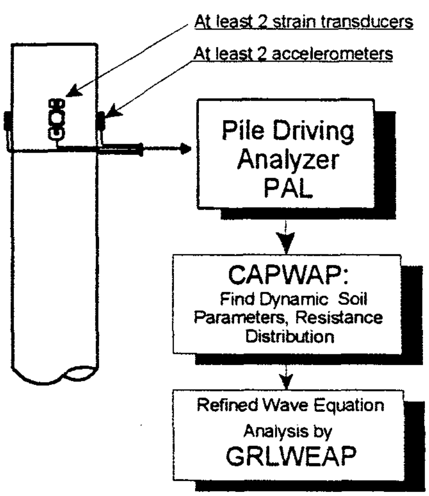


Figure 1. Block Diagram of Refined Wave Equation Analysis

yields the bearing capacity. This approach requires no measurements other than blow count. Rather it requires an accurate knowledge of the various parameters describing hammer, driving system, pile, and soil. The wave equation is also very useful during the design stage of a project for the selection of hammer, cushion, and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (Figure 1) is often performed by inputting the PDA and CAPWAP calculated parameters. With many of the dynamic parameters verified by the dynamic tests, it is a more reliable basis for a safe and sufficient driving criterion.

4.1.2 CASE METHOD

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force, F(t), and pile top velocity, v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \quad (1)$$

where:

t = a point in time after impact.
t_2 = time t + 2L/c.
L = pile length below gages.
c = $(E/\rho)^{1/2}$ is the speed of the stress wave.
ρ = pile mass density.
Z = EA/c is the pile impedance.
E = elastic modulus of the pile (ρ c^2).
A = pile cross sectional area.

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t). \quad (2)$$

The dynamic component may be computed from a soil damping factor, J, and the pile velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \quad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 could be evaluated. Most commonly, T is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2 therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed for one record. The capacities for other hammer blows are then quickly calculated for the thus selected Case Method and its associated damping factor.

The static resistance calculated by either Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDI-PLOT program or formerly in the DOS based PDAPLOT program.

4.1.3 CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors, and soil stiffness values. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters based on the dynamic measurements.

4.2 STRESSES

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance, R(t), minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = ½[F(t) - Zv(t)] \tag{4}$$

$$W_d = ½[F(t) + Zv(t)] \tag{5}$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

4.3 *PILE INTEGRITY BY PDA*

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{(E\rho)}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA:

$$\beta = (1 - \alpha)/(1 + \alpha) \qquad (6)$$

with

$$\alpha = \tfrac{1}{2}(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \qquad (7)$$

where

W_{UR} is the upward traveling wave at the onset of the damage reflected wave. It is caused by resistance.

W_{UD} is the upward traveling reflection wave due to the damage.

W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.

Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

4.4 *HAMMER PERFORMANCE BY PDA*

The PDA calculates the energy transferred to the pile top from:

$$E(t) = \int_0^t F(\tau)v(\tau)\,d\tau \qquad (8a)$$

The maximum of the E(t) curve is often called **ENTHRU**; it is the most important information for an overall evaluation of the performance of a hammer and driving system. **ENTHRU** or **EMX** allow for a classification of the hammer's performance when presented as, e_T, the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency.

$$e_T = EMX/E_R \qquad (8b)$$

where

E_R is the hammer manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\,T_B^2 - h_L \qquad (9)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

4.5 *DETERMINATION OF WAVE SPEED*

An important facet of dynamic pile testing is an assessment of pile material properties. Since, in most cases general force is determined from strain by multiplication with elastic modulus, **E**, and cross sectional area, **A**, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T. \qquad (10)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho. \qquad (11)$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c of the whole pile is lower than the wave speed at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes during the pile installation.
- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

5. *DATA QUALITY CHECKS*

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

5.1 PROPORTIONALITY

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c). \qquad (12a)$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \qquad (12b)$$

or strain

$$\varepsilon = v / c. \qquad (12c)$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

5.2 NUMBER OF SENSORS

Measurements are always taken at opposite sides of the pile so that the average force and velocity in the pile can be calculated. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. In that case the averaging of the two strain signals does not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

6. LIMITATIONS, ADDITIONAL CONSIDERATIONS

6.1 MOBILIZATION OF CAPACITY

Estimates of pile capacity from dynamic testing indicate thew **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

6.2 TIME DEPENDENT SOIL RESISTANCE EFFORTS

Static pile capacity from dynamic method calculations provides an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur as a result of soil setup and relaxation. Therefore, <u>restrike testing</u> **usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

6.2.1 SOIL SETUP

Because excess positive pore pressures often develop during pile driving in fine grained soils (clays, silts, or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomenon is routinely called soil setup or soil freeze. There are numerous other reasons for soil setup such as realignment of clay particles, arching that reduces effective stresses during pile installation in very dense sands, soil fatigue in over-consolidated clays, etc.

6.2.2 RELAXATION

Relaxation capacity reduction with time has been observed for piles driven into weathered shale, and may take several days to fully develop. Where relaxation occurs, pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically with particular emphasis on the first few blows. Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. In general, relaxation occurs at the pile toe and is therefore relevant for end bearing piles. Restrike tests should be performed and compared with the records from early restrike blows in order to avoid dangerous over-predictions.

6.3 CAPACITY RESULTS FOR OPEN PILE PROFILES

Open ended pipe piles or H-piles which do not bear on rock may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

6.4 CAPWAP ANALYSIS RESULTS

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the signal match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

6.5 STRESSES

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States it has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

- 90% of yield strength for steel piles
- 85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression
- 100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension
- 70% of the reinforcement strength for regularly reinforced concrete piles in tension
- 300% of the static design allowable stress for timber.

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements. The above allowable stresses also apply to those calculated by wave equation.

6.6 ADDITIONAL DESIGN CONSIDERATIONS

Numerous factors have to be considered in pile foundation design. Some of these considerations include:

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements,
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,
- loss of shaft resistance due to scour or other effects,
- loss of structural pile strength due to additional bending loads, buckling (the dynamic loads generally do not cause buckling even though they may exceed the buckling strength of the pile section), corrosion, etc.

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

6.7 WAVE EQUATION ANALYSIS RESULTS

The results calculated by the wave equation analysis program depend on a variety of hammer, pile, and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

7. FACTORS OF SAFETY

Run to failure, static or dynamic load tests yield an ultimate pile bearing capacity, R_{ult}. If this failure load were applied to the pile, then excessive settlements would occur. Therefore, it is absolutely necessary that the actually applied load, also called the design load, R_d (or working load or safe load), is less than R_{ult}. In most soils, to limit settlements it is necessary that R_{ult}, is at least 50% higher than R_d. This means that

$$R_{ult} \geq 1.5 \, R_d,$$

or the Factor of Safety has to be at least 1.5.

Unfortunately, neither applied loads nor R_{ult} are exactly known. One static load test may be performed at a site, but that would not guarantee that all other piles have the same capacity and it is to be expected that a certain percentage of the production piles have lower capacities, either due to soil variability or due to pile damage. If, for example, dynamic pile tests are performed on piles in shale only a short time after pile installation, then the test capacity may be higher than the long term capacity of the pile. On the other hand, due to soil setup, piles generally gain capacity after installation and since tests are only done a short time after installation, a lower capacity value is ascertained than the capacity that eventually develops.

Not only are bearing capacity values of all piles unknown, even loads vary considerably and occasional overloads must be expected. We would not want a structure to become unserviceable or useless because of either an occasional overload or a few piles with low capacity. For this reason, and to avoid being overly conservative which would mean excessive cost, modern safety concepts suggest that the overall factor of safety should reflect the uncertainty in both loads and resistance. Thus, if all piles were statically tested and if we carefully controlled the loads, we probably could live with F.S. = 1.5. However, in general, depending on the building type or load combinations and as a function of quality assurance of pile foundations, a variety of Factors of Safety have been proposed.

For example, based on AASHTO specifications for highway related loads, the Federal Highway Administration proposes the following:

F.S.= 2.00 for static load test with wave equation.
F.S.= 2.25 for dynamic testing with wave equation analysis.
F.S.= 2.50 for indicator piles with wave equation analysis.
F.S.= 2.75 for wave equation analysis.
F.S.= 3.50 for Gates or other dynamic formula.

It should be mentioned that all of these methods should always be combined with soil exploration and static pile analysis. Also, specifications are occasionally updated and therefore the latest version should be variously consulted for the appropriate factors of safety.

Codes, among them PDCA, ASCE, or specifications issued by State Departments of Transportation specify different factors of safety. However, the range of recommended overall factors of safety in the United States varies between 1.9 and 6.

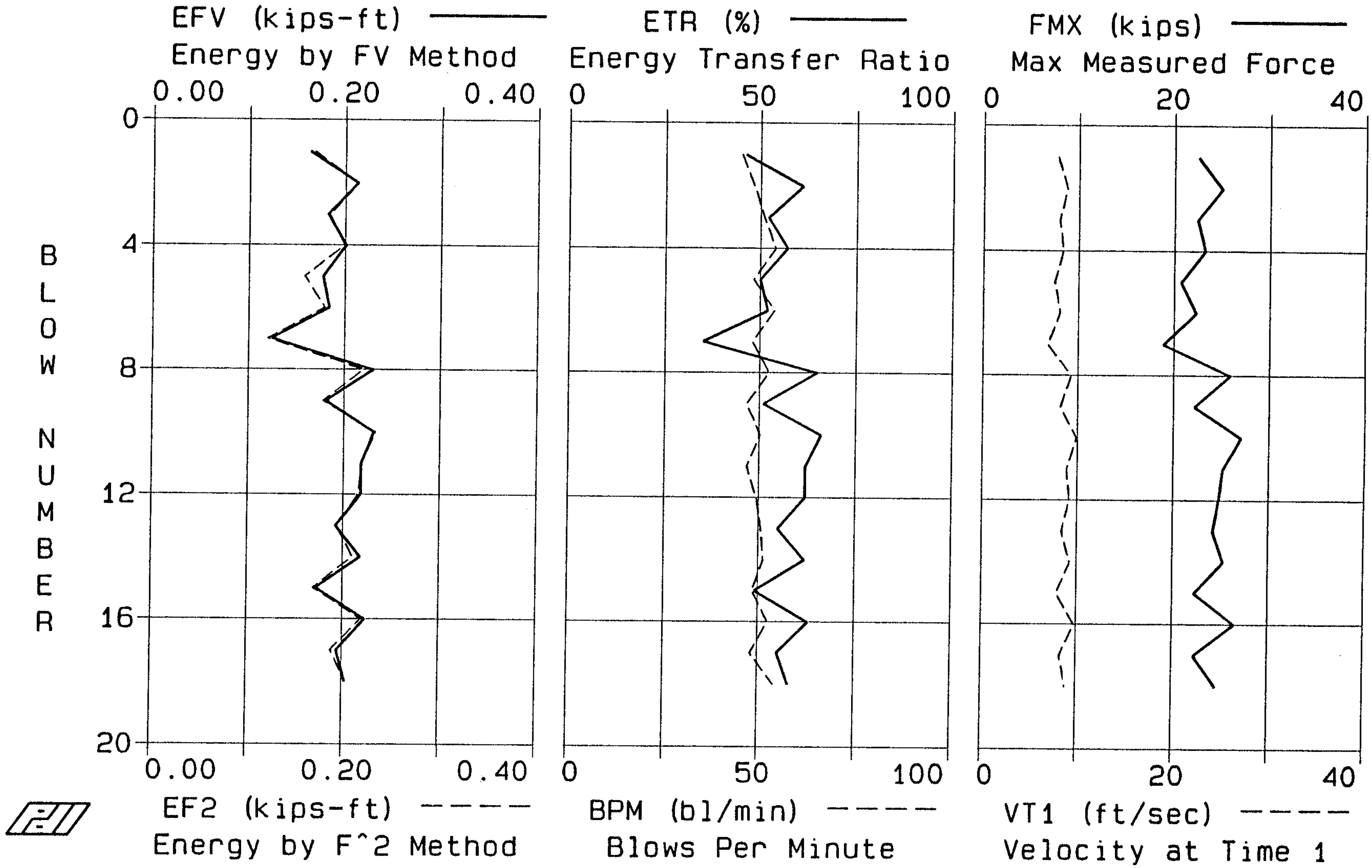
It is the designer's responsibility to identify design loads together with the adopted safety factor concept and associated construction control procedure. The required factors of safety should be included in design drawings or specifications together with the required testing. Only contractors bid for the work and develop the most economical solution. This should include a program of increased testing for lower required pile capacities. This will also help to reduce the confusion that often exists on construction sites as to design loads and require capacities. In any event, it be cannot expected that the test engineer is aware of and responsible for the variety of considerations that must be met to find the appropriate factor of safety.

EXHIBIT B

Dynamic Testing Field Results

GRL & Associates, Inc. 02-Aug-02

Clinton Power Station, Boring 2 @ 43.5-45.0 Ft., GP 7500 - Cathead & Rope



Pile: Boring 2 @ 43.5-45.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 49.5 ft

Proj: Clinton Power Station Pg1
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
5	10*	44.00	AVG	5	0.187	0.184	53	22.8	8.1	49.3
			STD	5	0.020	0.022	6	1.6	0.5	3.3
			MAX	5	0.213	0.213	61	25.0	8.8	53.9
			MIN	5	0.163	0.158	46	20.7	7.5	45.0
10	10	44.50	AVG	5	0.190	0.186	54	23.3	8.5	50.2
			STD	5	0.044	0.044	13	3.3	1.2	3.0
			MAX	5	0.231	0.233	66	27.1	9.9	53.7
			MIN	5	0.124	0.120	35	18.9	6.8	46.6
18	16	45.00	AVG	8	0.205	0.202	58	24.4	8.8	50.1
			STD	8	0.018	0.018	5	1.5	0.6	2.5
			MAX	8	0.223	0.219	63	26.5	9.8	54.2
			MIN	8	0.172	0.169	49	22.3	7.9	46.7

*BLC USER INPUT

BL#	COMMENTS
1	Operator: Francisco
1	Reported Blow Count: 5, 5, 8.

DRIVEN (02-Aug-02 : B-2-435.MDF)

Pile: Boring 2 @ 43.5-45.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 49.5 ft

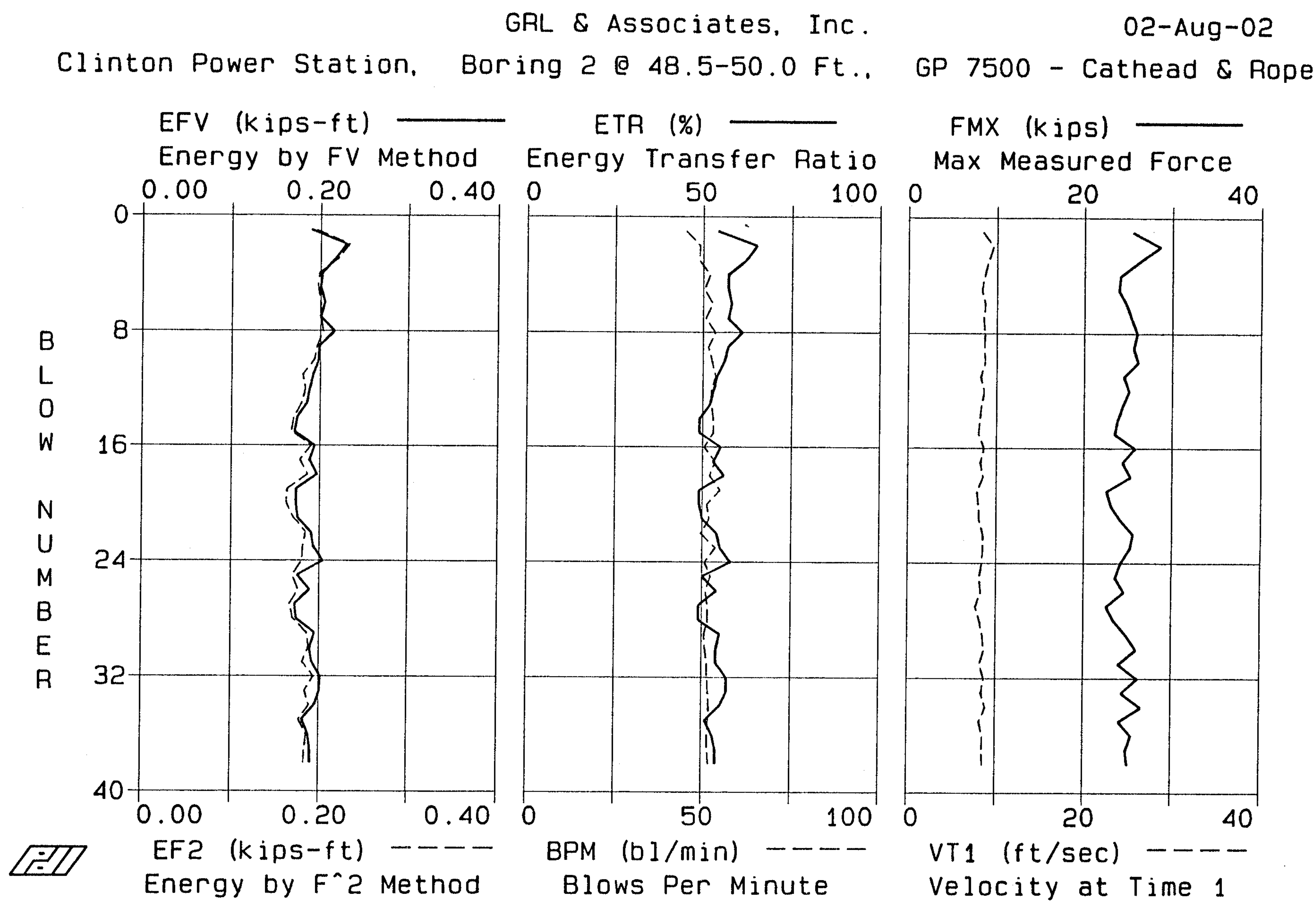
Proj: Clinton Power Station Pg1
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
18	16	45.00	AVG	13	0.199	0.196	57	24.0	8.7	50.1
			STD	13	0.030	0.030	9	2.3	0.9	2.6
			MAX	13	0.231	0.233	66	27.1	9.9	54.2
			MIN	13	0.124	0.120	35	18.9	6.8	46.6

DRIVEN (02-Aug-02 : B-2-435.MDF)



GRL & Associates, Inc.
02-Aug-02
Clinton Power Station, Boring 2 @ 48.5-50.0 Ft., GP 7500 - Cathead & Rope
EFV (kips-ft)
Energy by FV Method
ETR (%)
Energy Transfer Ratio
FMX (kips)
Max Measured Force
BLOW NUMBER
EF2 (kips-ft)
Energy by F^2 Method
BPM (bl/min)
Blows Per Minute
VT1 (ft/sec)
Velocity at Time 1

Pile: Boring 2 @ 48.5-50.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 54.5 ft

Proj: Clinton Power Station Pg1
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
2	2*	49.00	AVG	2	0.210	0.214	60	27.1	9.0	47.0
			STD	2	0.029	0.027	8	2.2	0.8	2.8
			MAX	2	0.230	0.233	65	28.6	9.6	49.0
			MIN	2	0.189	0.195	54	25.5	8.4	45.0
17	30	49.50	AVG	15	0.195	0.190	55	24.9	8.5	52.0
			STD	15	0.013	0.014	4	0.9	0.3	1.4
			MAX	15	0.217	0.220	62	26.3	9.0	53.5
			MIN	15	0.171	0.167	49	23.5	8.0	48.7
38	42	50.00	AVG	21	0.188	0.179	53	24.6	8.4	51.7
			STD	21	0.010	0.009	3	1.1	0.3	1.1
			MAX	21	0.204	0.194	58	26.6	8.9	55.0
			MIN	21	0.172	0.162	49	22.6	7.7	49.5

*BLC USER INPUT

BL# COMMENTS
1 Operator: Dave
1 Reported Blow Count: 9, 15, 21.

DRIVEN (02-Aug-02 : B-2-485.MDF)

Pile: Boring 2 @ 48.5-50.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 54.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

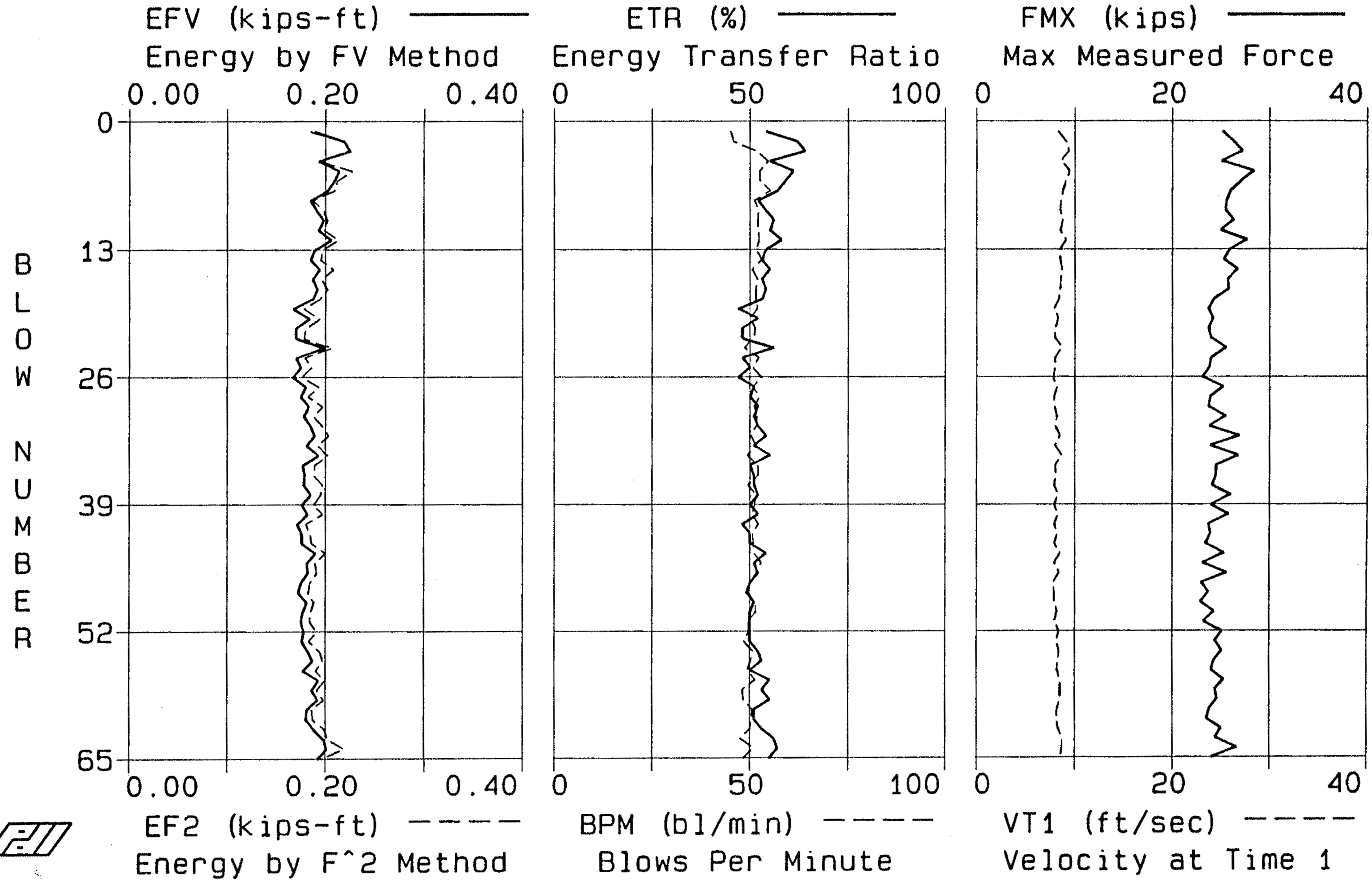
FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
38	42	50.00	AVG	36	0.191	0.184	54	24.7	8.4	51.8
			STD	36	0.012	0.013	3	1.0	0.3	1.2
			MAX	36	0.217	0.220	62	26.6	9.0	55.0
			MIN	36	0.171	0.162	49	22.6	7.7	48.7

DRIVEN (02-Aug-02 : B-2-485.MDF)

GRL & Associates, Inc. 02-Aug-02

Clinton Power Station, Boring 2 @ 53.5-55.0 Ft., GP 7500 - Cathead & Rope



Pile: Boring 2 @ 53.5-55.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 59.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
15	30*	54.00	AVG	15	0.199	0.205	57	26.2	8.8	51.4
			STD	15	0.013	0.013	4	1.0	0.4	2.7
			MAX	15	0.225	0.227	64	28.4	9.5	55.1
			MIN	15	0.185	0.185	52	25.0	8.3	45.0
65	119	54.42	AVG	50	0.182	0.191	51	24.5	8.2	50.6
			STD	50	0.008	0.009	2	1.0	0.3	1.3
			MAX	50	0.201	0.219	57	26.9	8.7	52.9
			MIN	50	0.167	0.176	47	22.9	7.8	47.5

*BLC USER INPUT

BL#	COMMENTS
1	Operator: Greg
1	Reported Blow Count: 22, 50/5 inches

DRIVEN (02-Aug-02 : B-2-535.MDF)

Pile: Boring 2 @ 53.5-55.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 59.5 ft

Proj: Clinton Power Station

SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

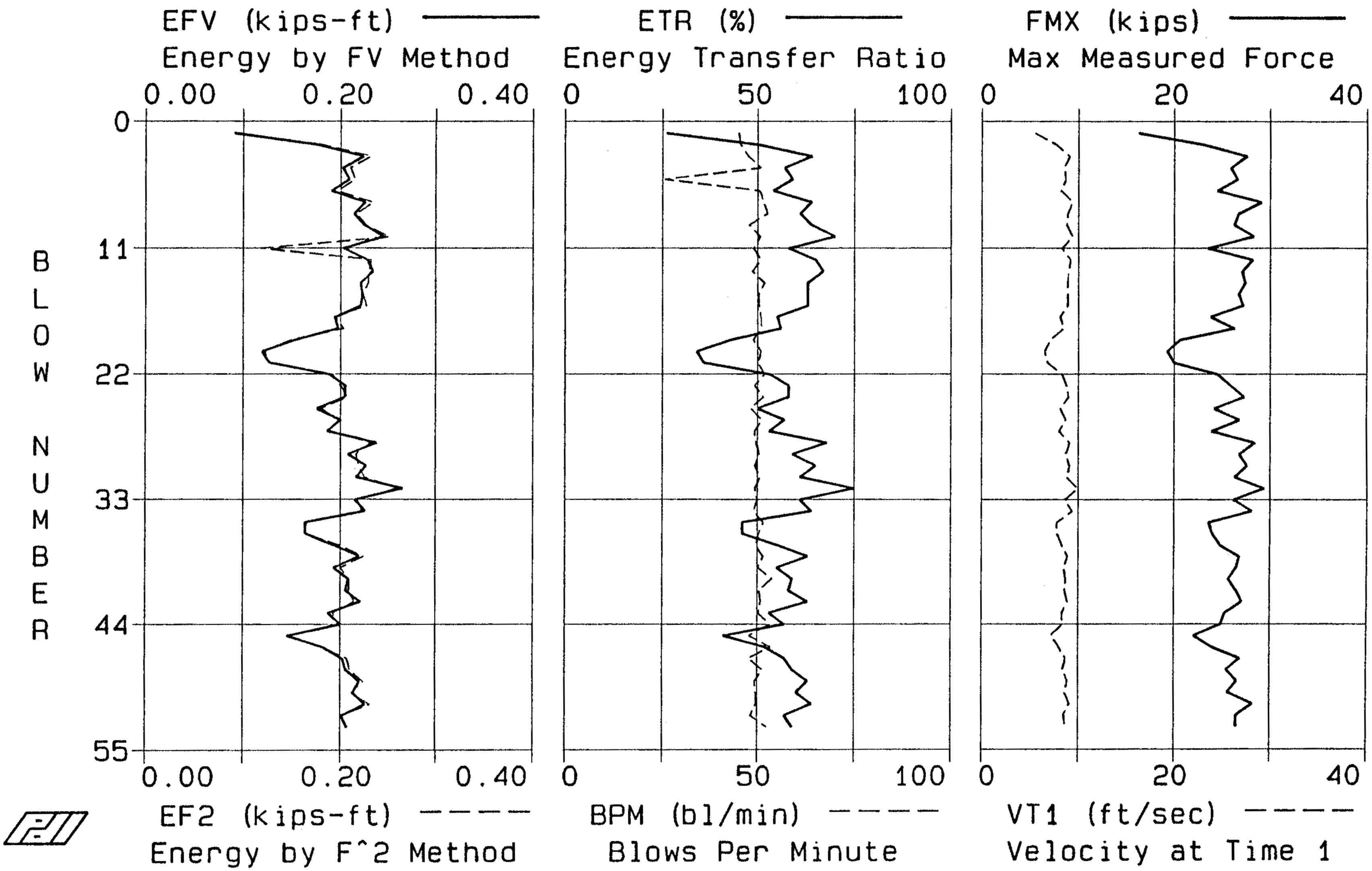
BL# end	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
65	AVG	50	0.182	0.191	51	24.5	8.2	50.6
	STD	50	0.008	0.009	2	1.0	0.3	1.3
	MAX	50	0.201	0.219	57	26.9	8.7	52.9
	MIN	50	0.167	0.176	47	22.9	7.8	47.5

DRIVEN (12-Aug-02 : B-2-535.MDF)

ACTUAL DATE 02-Aug-02
(Correction by CH2MHILL)

GRL & Associates, Inc. 02-Aug-02

Clinton Power Station, Boring 2 @ 58.5-60.0 Ft., GP 7500 - Cathead & Rope



Pile: Boring 2 @ 58.5-60.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 64.5 ft

Proj: Clinton Power Station

SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
23	46*	59.00	AVG	23	0.196	0.196	56	25.0	8.3	48.7
			STD	23	0.039	0.044	11	3.3	1.0	5.4
			MAX	23	0.245	0.251	70	29.1	9.4	52.5
			MIN	23	0.091	0.096	26	16.3	5.5	25.6
37	28	59.50	AVG	14	0.206	0.207	58	26.2	8.7	49.9
			STD	14	0.029	0.027	8	1.9	0.6	1.0
			MAX	14	0.265	0.259	75	29.4	10.0	51.5
			MIN	14	0.163	0.162	46	23.6	7.8	48.1
53	32	60.00	AVG	16	0.202	0.204	58	25.9	8.5	50.6
			STD	16	0.020	0.020	6	1.4	0.4	1.9
			MAX	16	0.226	0.231	64	28.2	9.1	53.7
			MIN	16	0.145	0.146	41	22.1	7.2	47.7

*BLC USER INPUT

BL#	COMMENTS
1	Operator: Francisco
1	Reported Blow Count: 21, 14, 16.

DRIVEN (02-Aug-02 : B-2-585.MDF)

Pile: Boring 2 @ 58.5-60.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 64.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

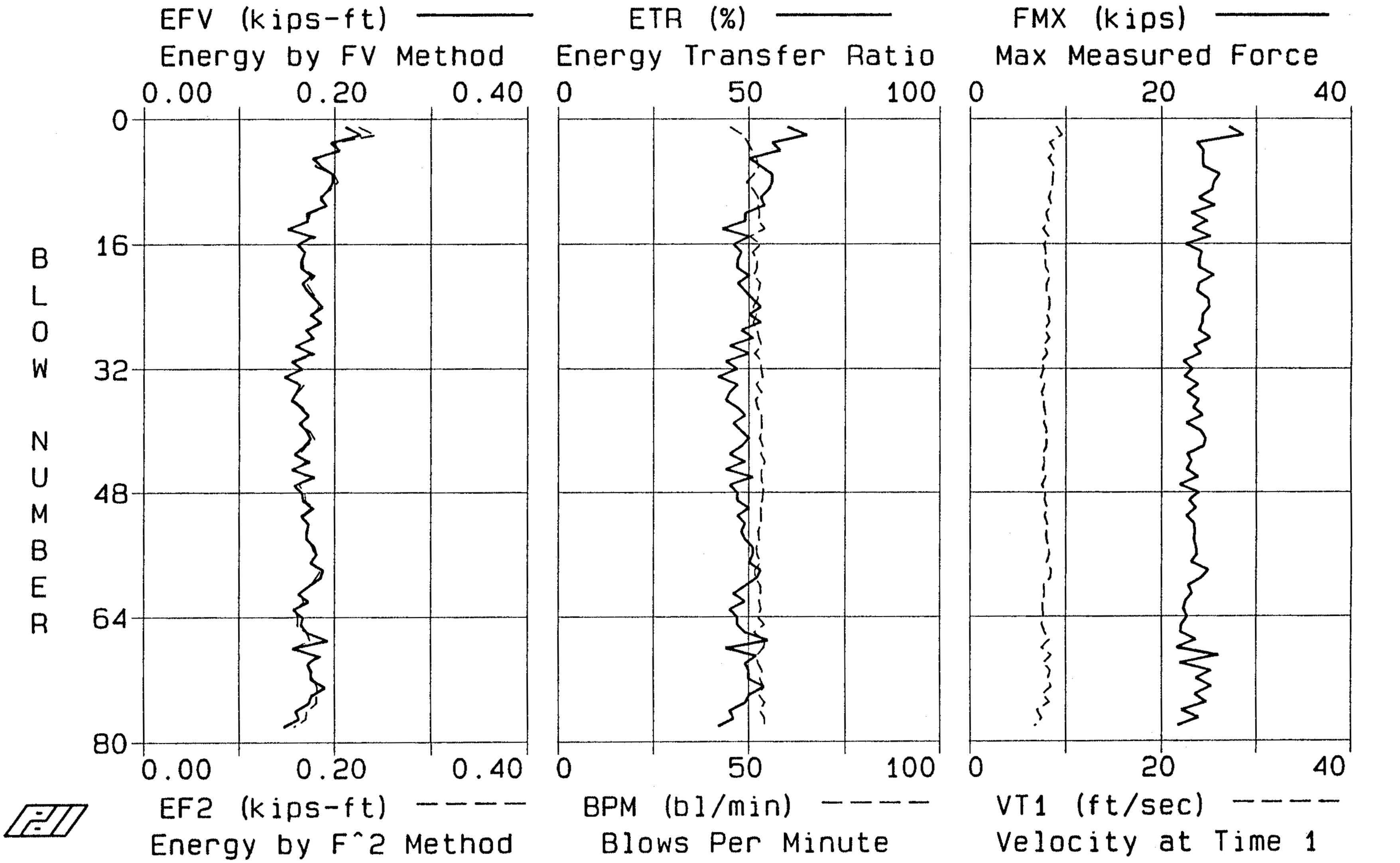
BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
53	32	60.00	AVG	30	0.204	0.205	58	26.1	8.6	50.3
			STD	30	0.024	0.023	7	1.6	0.5	1.6
			MAX	30	0.265	0.259	75	29.4	10.0	53.7
			MIN	30	0.145	0.146	41	22.1	7.2	47.7

DRIVEN (12-Aug-02 : B-2-585.MDF)

ACTUAL DATE 02-Aug-02
(Correction by CH2MHILL)

GRL & Associates, Inc. 02-Aug-02

Clinton Power Station, Boring 2 @ 63.5-65.0 Ft., GP 7500 - Cathead & Rope



Pile: Boring 2 @ 63.5-65.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 69.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
28	56*	64.00	AVG	28	0.182	0.184	52	24.6	8.3	51.4
			STD	28	0.017	0.019	5	1.2	0.4	1.7
			MAX	28	0.227	0.242	65	28.6	9.6	54.0
			MIN	28	0.151	0.151	43	22.5	7.6	45.0
78	119	64.42	AVG	50	0.169	0.170	48	23.3	7.8	53.0
			STD	50	0.010	0.008	3	1.0	0.4	0.8
			MAX	50	0.193	0.186	55	26.0	8.5	54.3
			MIN	50	0.147	0.149	42	21.6	6.8	51.4

*BLC USER INPUT

BL# COMMENTS
1 Operator: Dave
1 Reported Blow Count: 32, 50/5 inches.

DRIVEN (02-Aug-02 : B-2-635.MDF)

Pile: Boring 2 @ 63.5-65.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 69.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
78	AVG	50	0.169	0.170	48	23.3	7.8	53.0
	STD	50	0.010	0.008	3	1.0	0.4	0.8
	MAX	50	0.193	0.186	55	26.0	8.5	54.3
	MIN	50	0.147	0.149	42	21.6	6.8	51.4

DRIVEN (12-Aug-02 : B-2-635.MDF)

ACTUAL DATE 02-Aug-02
(Correction by CH2MHILL)

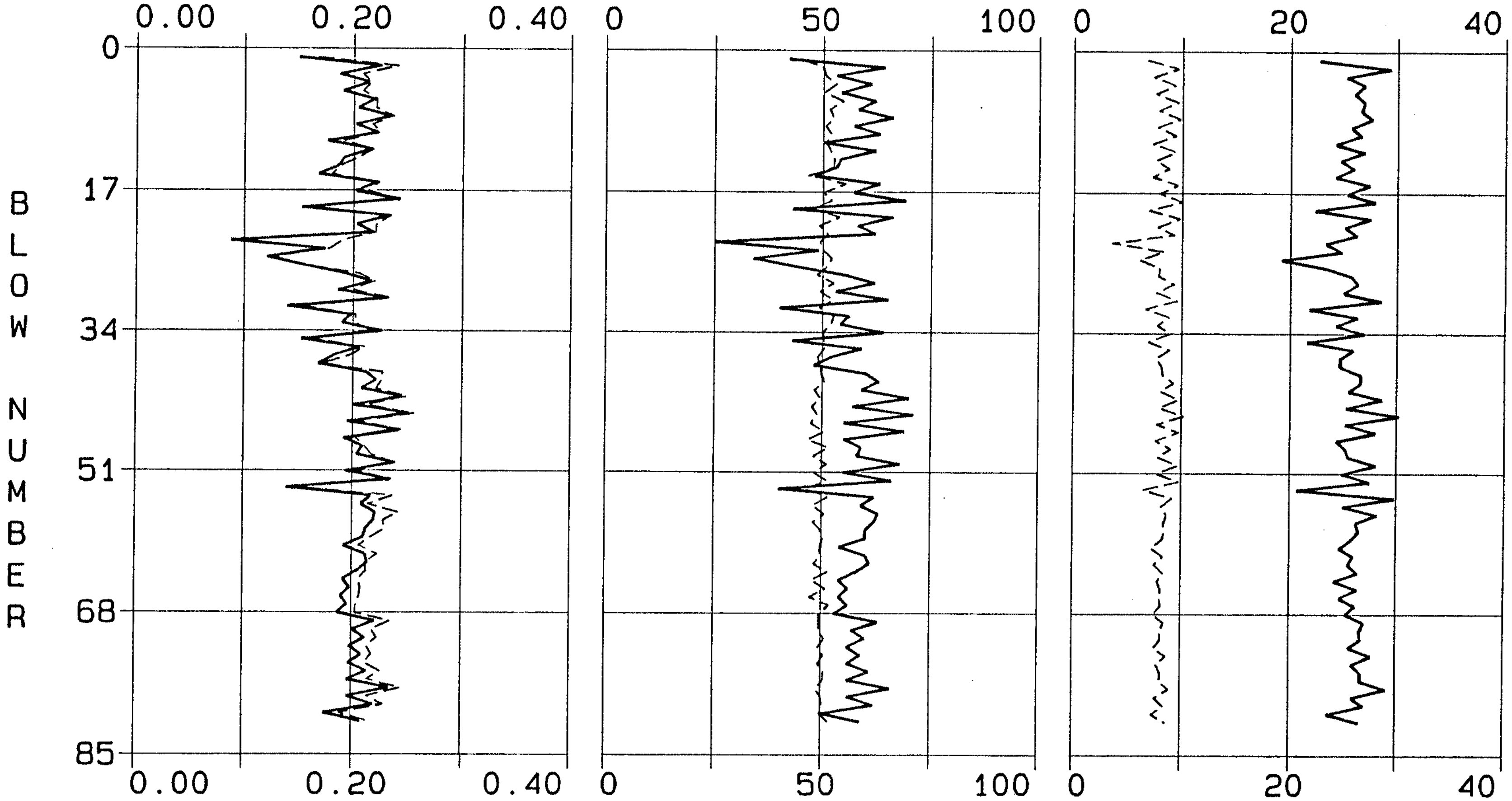
GRL & Associates, Inc. 02-Aug-02

Clinton Power Station, Boring 2 @ 68.5-70.0 Ft., GP 7500 - Cathead & Rope

EFV (kips-ft) ——— Energy by FV Method

ETR (%) ——— Energy Transfer Ratio

FMX (kips) ——— Max Measured Force



EF2 (kips-ft) - - - - Energy by F^2 Method

BPM (bl/min) - - - - Blows Per Minute

VT1 (ft/sec) - - - - Velocity at Time 1

Pile: Boring 2 @ 68.5-70.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 74.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
28	56*	69.00	AVG	28	0.193	0.205	55	25.5	8.2	50.8
			STD	28	0.036	0.028	10	2.1	1.4	2.3
			MAX	28	0.242	0.242	69	29.3	10.0	55.2
			MIN	28	0.088	0.127	25	19.2	3.6	45.0
53	50	69.50	AVG	25	0.203	0.210	58	25.7	8.5	49.5
			STD	25	0.031	0.028	9	2.2	1.0	1.7
			MAX	25	0.251	0.256	71	30.2	10.2	52.5
			MIN	25	0.140	0.147	40	20.7	6.6	45.8
81	56	70.00	AVG	28	0.206	0.218	58	26.3	8.2	49.8
			STD	28	0.013	0.013	4	1.3	0.5	1.2
			MAX	28	0.234	0.245	66	29.8	9.2	51.8
			MIN	28	0.175	0.189	50	23.6	7.3	47.3

*BLC USER INPUT

BL# COMMENTS
1 Operator: Francisco
1 Reported Blow Count: 19, 25, 28.

DRIVEN (02-Aug-02 : B-2-685.MDF)

Pile: Boring 2 @ 68.5-70.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 74.5 ft

Proj: Clinton Power Station Pg1
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

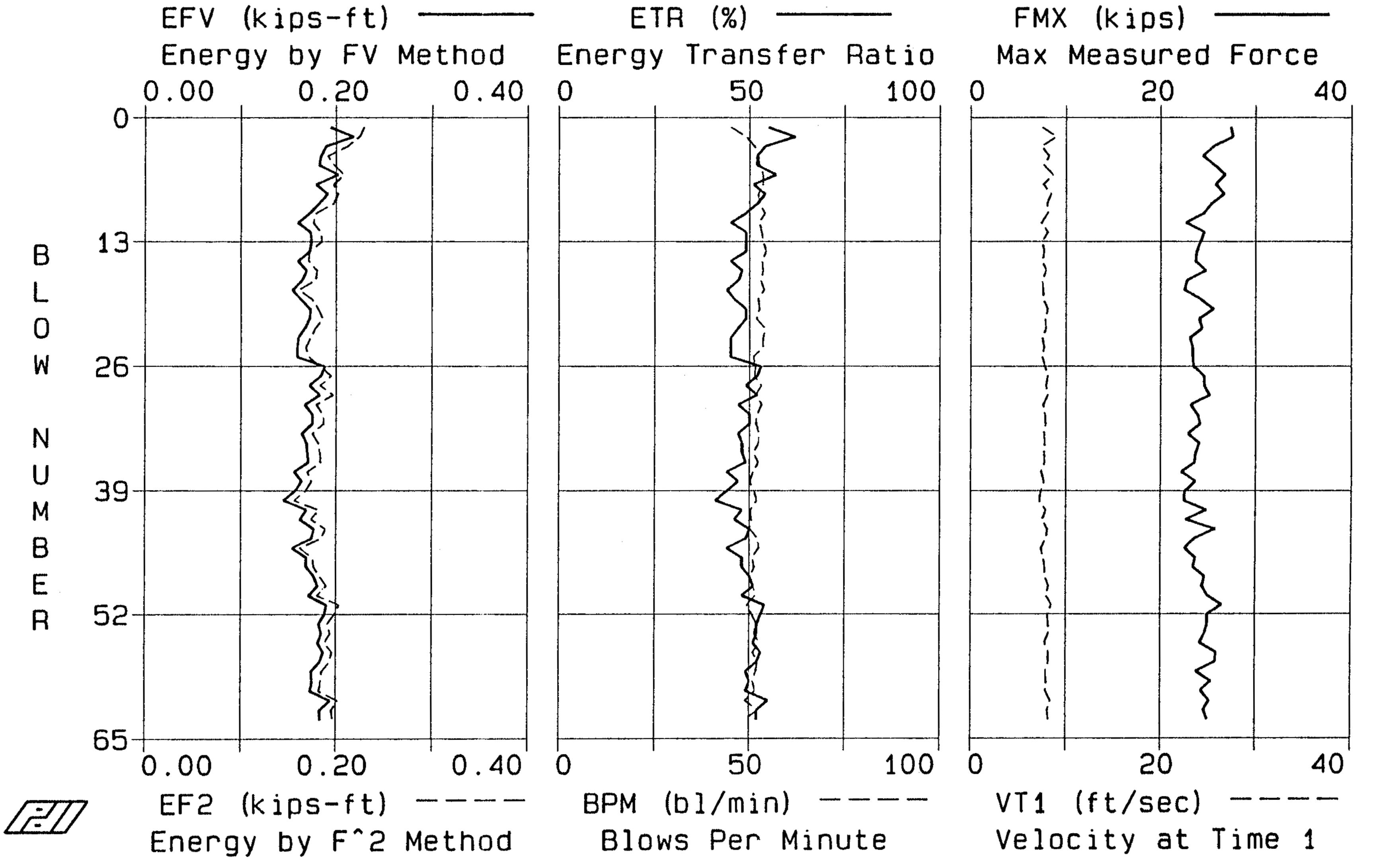
BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
81	56	70.00	AVG	53	0.204	0.214	58	26.0	8.3	49.7
			STD	53	0.023	0.022	6	1.8	0.8	1.5
			MAX	53	0.251	0.256	71	30.2	10.2	52.5
			MIN	53	0.140	0.147	40	20.7	6.6	45.8

DRIVEN (12-Aug-02 : B-2-685.MDF)

ACTUAL DATE 02-Aug-02
(Correction by CH2MHILL)

GRL & Associates, Inc. 02-Aug-02

Clinton Power Station, Boring 2 @ 73.5-75.0 Ft., GP 7500 - Cathead & Rope



Pile: Boring 2 @ 73.5-75.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 79.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
15	30*	74.00	AVG	15	0.182	0.195	52	25.3	7.9	52.2
			STD	15	0.015	0.018	4	1.4	0.5	2.3
			MAX	15	0.218	0.229	62	27.6	9.0	54.3
			MIN	15	0.160	0.171	45	22.7	7.4	45.0
40	50	74.50	AVG	25	0.167	0.178	47	23.7	7.7	52.2
			STD	25	0.010	0.009	3	0.9	0.2	1.1
			MAX	25	0.188	0.197	53	25.6	8.1	54.0
			MIN	25	0.145	0.156	41	22.2	7.2	50.0
63	46	75.00	AVG	23	0.177	0.187	50	24.6	8.0	51.0
			STD	23	0.009	0.010	3	1.0	0.3	1.1
			MAX	23	0.194	0.203	55	26.5	8.5	52.6
			MIN	23	0.154	0.162	44	22.6	7.4	48.9

*BLC USER INPUT

BL#	COMMENTS
1	Operator: Dave
1	Reported Blow Count: 15, 25, 23.

DRIVEN (02-Aug-02 : B-2-735.MDF)

Pile: Boring 2 @ 73.5-75.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 79.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

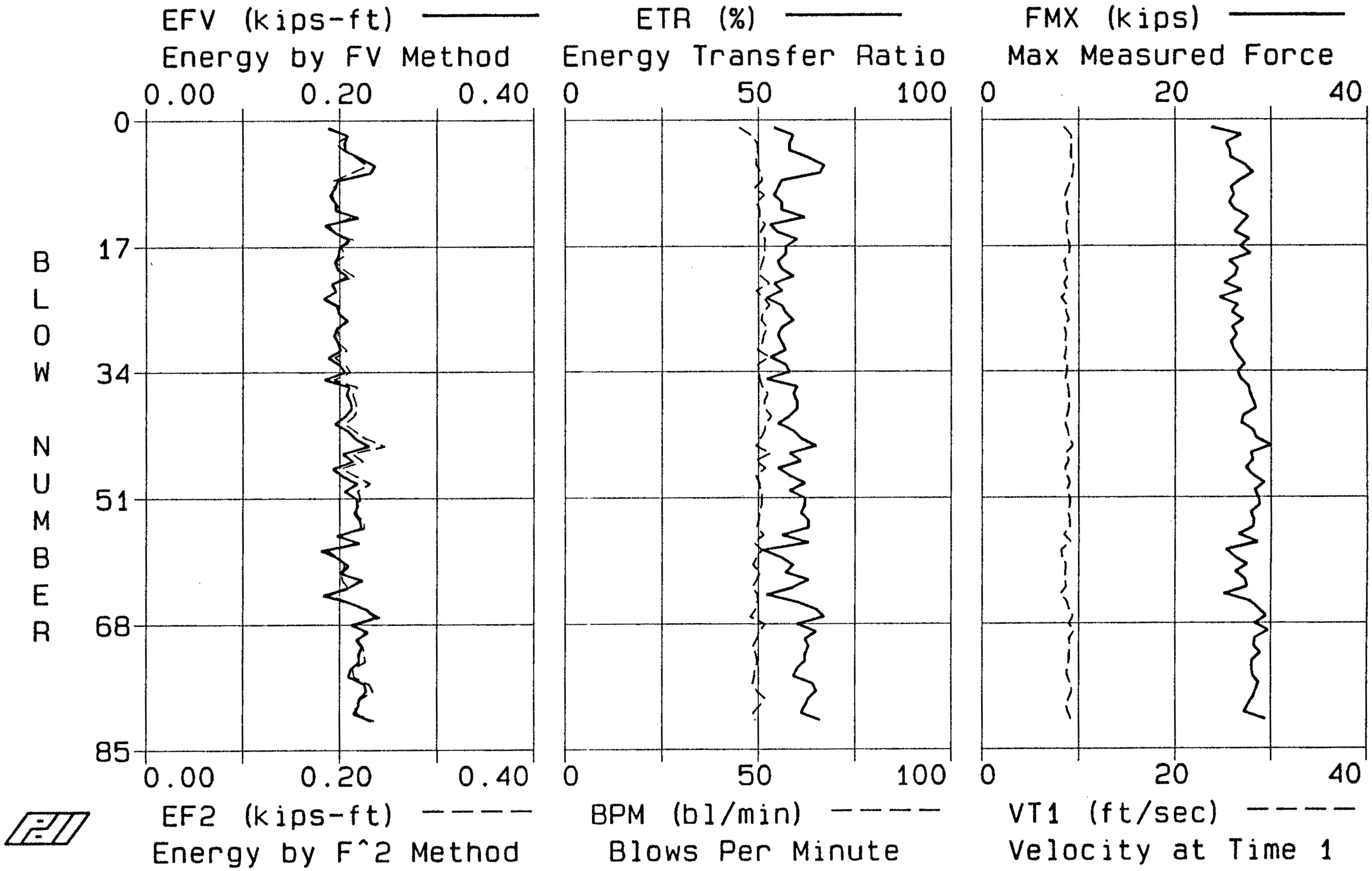
BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
63	46	75.00	AVG	48	0.172	0.182	49	24.2	7.9	51.6
			STD	48	0.011	0.011	3	1.0	0.3	1.2
			MAX	48	0.194	0.203	55	26.5	8.5	54.0
			MIN	48	0.145	0.156	41	22.2	7.2	48.9

DRIVEN (12-Aug-02 : B-2-735.MDF)

ACTUAL DATE 02-Aug-02
(Correction by CH2MHILL)

GRL & Associates, Inc. 02-Aug-02

Clinton Power Station, Boring 2 @ 78.5-80.0 Ft., GP 7500 - Cathead & Rope



Pile: Boring 2 @ 78.5-80.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 84.5 ft

Proj: Clinton Power Station Pg1
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
14	28*	79.00	AVG	14	0.205	0.202	58	26.3	9.0	49.6
			STD	14	0.016	0.012	4	1.1	0.3	1.7
			MAX	14	0.236	0.228	67	28.1	9.5	51.7
			MIN	14	0.185	0.188	53	23.8	8.5	45.0
41	54	79.50	AVG	27	0.200	0.204	57	26.7	8.8	51.2
			STD	27	0.008	0.009	2	0.9	0.2	1.0
			MAX	27	0.212	0.218	60	28.4	9.1	53.2
			MIN	27	0.184	0.186	52	24.6	8.2	49.3
81	80	80.00	AVG	40	0.214	0.218	61	28.0	8.9	49.9
			STD	40	0.012	0.013	4	1.0	0.3	1.1
			MAX	40	0.237	0.246	67	30.0	9.5	53.0
			MIN	40	0.181	0.188	51	25.1	8.1	47.8

*BLC USER INPUT

BL# COMMENTS
1 Operator: Greg
1 Reported Blow Count: 15, 27, 40.

DRIVEN (02-Aug-02 : B-2-785.MDF)

Pile: Boring 2 @ 78.5-80.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 84.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

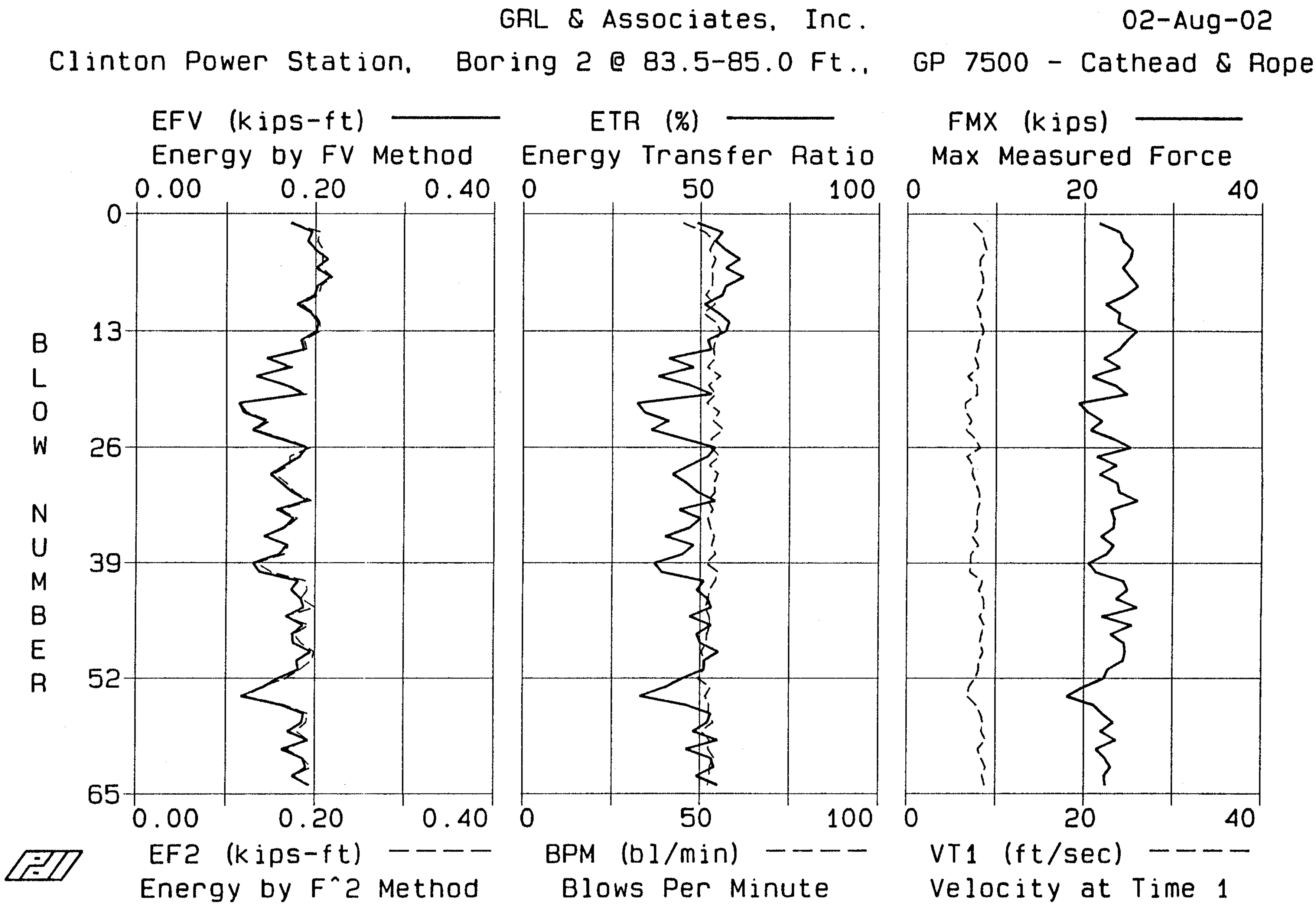
EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
81	80	80.00	AVG	67	0.208	0.212	59	27.5	8.8	50.4
			STD	67	0.013	0.013	4	1.2	0.3	1.3
			MAX	67	0.237	0.246	67	30.0	9.5	53.2
			MIN	67	0.181	0.186	51	24.6	8.1	47.8

DRIVEN (12-Aug-02 : B-2-785.MDF)

ACTUAL DATE 02-Aug-02
(Correction by CH2MHILL)


GRL & Associates, Inc.
02-Aug-02
Clinton Power Station, Boring 2 @ 83.5-85.0 Ft., GP 7500 - Cathead & Rope
EFV (kips-ft)
Energy by FV Method
ETR (%)
Energy Transfer Ratio
FMX (kips)
Max Measured Force
0.00 0.20 0.40
0 50 100
0 20 40
BLOW NUMBER
0
13
26
39
52
65
EF2 (kips-ft)
Energy by F^2 Method
BPM (bl/min)
Blows Per Minute
VT1 (ft/sec)
Velocity at Time 1

Pile: Boring 2 @ 83.5-85.0 Ft. Proj: Clinton Power Station Pg1
Info: GP 7500 - Cathead & Rope SP: 0.492 k/ft^3
AR: 1.5 in^2 WS: 16808 ft/s
LE: 89.5 ft EM: 30000 KSI

EFV: Energy by FV Method FMX: Max Measured Force
EF2: Energy by F^2 Method VT1: Velocity at Time 1
ETR: Energy Transfer Ratio BPM: Blows Per Minute

BL# end	bl/ft	depth ft	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
14	28*	84.00	AVG	14	0.197	0.201	56	24.4	8.3	52.5
			STD	14	0.012	0.012	4	1.2	0.4	2.6
			MAX	14	0.217	0.219	62	26.0	8.9	55.5
			MIN	14	0.172	0.172	49	21.7	7.4	45.0
64	119	84.42	AVG	50	0.166	0.170	47	22.8	7.9	52.8
			STD	50	0.022	0.022	6	1.7	0.7	1.4
			MAX	50	0.195	0.200	55	26.0	8.9	56.3
			MIN	50	0.114	0.114	32	18.1	6.5	49.3

*BLC USER INPUT

BL# COMMENTS
1 Operator: Francisco
1 Reported Blow Count: 22, 50/5 inches.

DRIVEN (02-Aug-02 : B-2-835.MDF)

Pile: Boring 2 @ 83.5-85.0 Ft.
Info: GP 7500 - Cathead & Rope
AR: 1.5 in^2
LE: 89.5 ft

Proj: Clinton Power Station
SP: 0.492 k/ft^3
WS: 16808 ft/s
EM: 30000 KSI

EFV: Energy by FV Method
EF2: Energy by F^2 Method
ETR: Energy Transfer Ratio

FMX: Max Measured Force
VT1: Velocity at Time 1
BPM: Blows Per Minute

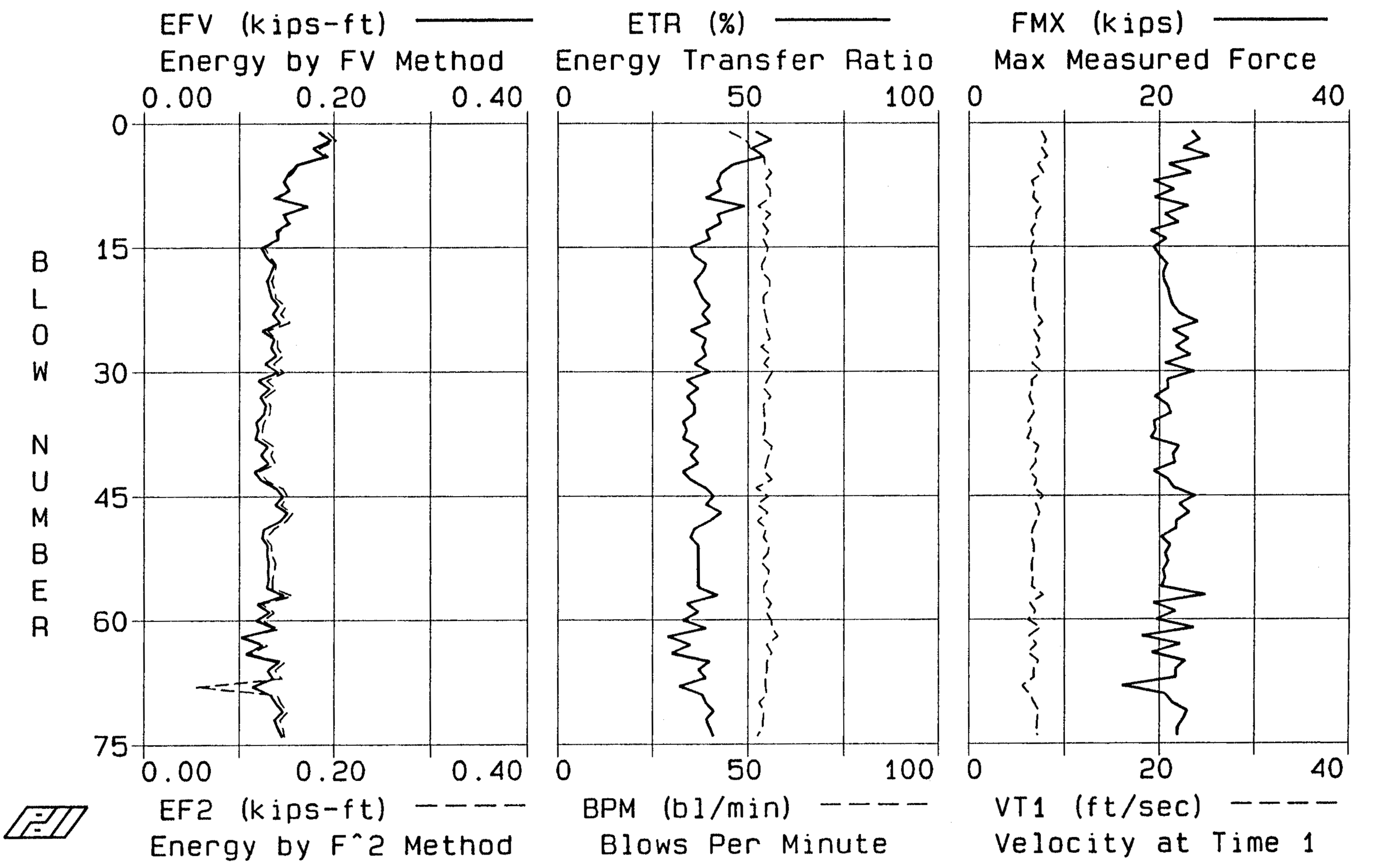
BL# end	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
64	AVG	50	0.166	0.170	47	22.8	7.9	52.8
	STD	50	0.022	0.022	6	1.7	0.7	1.4
	MAX	50	0.195	0.200	55	26.0	8.9	56.3
	MIN	50	0.114	0.114	32	18.1	6.5	49.3

DRIVEN (12-Aug-02 : B-2-835.MDF)

ACTUAL DATE 02-Aug-02
(Correction by CH2MHILL)

GRL & Associates, Inc. 02-Aug-02

Clinton Power Station, Boring 2 @ 88.5-90.0 Ft., GP 7500 - Cathead & Rope



Pile: Boring 2 @ 88.5-90.0 Ft. | Proj: Clinton Power Station | Pg1
Info: GP 7500 - Cathead & Rope | SP: 0.492 k/ft^3
AR: 1.5 in^2 | WS: 16808 ft/s
LE: 94.5 ft | EM: 30000 KSI

EFV: Energy by FV Method | FMX: Max Measured Force
EF2: Energy by F^2 Method | VT1: Velocity at Time 1
ETR: Energy Transfer Ratio | BPM: Blows Per Minute

BL#	TYPE	#Bls	EFV	EF2	ETR	FMX	VT1	BPM
end			kips-ft	kips-ft	%	kips	ft/sec	bl/min
74	AVG	74	0.136	0.141	39	21.3	6.9	54.3
	STD	74	0.017	0.019	5	1.6	0.5	1.7
	MAX	74	0.197	0.202	56	25.2	8.2	57.7
	MIN	74	0.102	0.055	29	16.1	5.6	45.0

*BLC USER INPUT

BL#	COMMENTS
1	Operator: Dave
1	Reported Blow Count: 100/4 inches.

DRIVEN (02-Aug-02 : B-2-885.MDF)

Pile:	Boring 2 @ 88.5-90.0 Ft.	Proj:	Clinton Power Station
Info:	GP 7500 - Cathead & Rope	SP:	0.492 k/ft^3
AR:	1.5 in^2	WS:	16808 ft/s
LE:	94.5 ft	EM:	30000 KSI

EFV:	Energy by FV Method	FMX:	Max Measured Force
EF2:	Energy by F^2 Method	VT1:	Velocity at Time 1
ETR:	Energy Transfer Ratio	BPM:	Blows Per Minute

BL# end	TYPE	#Bls	EFV kips-ft	EF2 kips-ft	ETR %	FMX kips	VT1 ft/sec	BPM bl/min
74	AVG	74	0.136	0.141	39	21.3	6.9	54.3
	STD	74	0.017	0.019	5	1.6	0.5	1.7
	MAX	74	0.197	0.202	56	25.2	8.2	57.7
	MIN	74	0.102	0.055	29	16.1	5.6	45.0

DRIVEN (12-Aug-02 : B-2-885.MDF)

ACTUAL DATE 02-Aug-02
(Correction by CH2MHILL)

EXHIBIT C

Strain Transducer

and

Accelerometer Calibration Information

Goble Rausche Likins and Associates, Inc.
CALIBRATION SHEET

Job Name Clinton Power Station	
Date August 2, 2002	Job# 027049

Strain Transducers

Sensor Number	Sensor Calibration	Calibration Date	Used ?
53 NWJ-1	218.12	7-10-01	✓
53 NWJ-2	218.54	7-10-01	✓

Piezoelectric Accelerometers

Sensor Number	Sensor Calibration	Calibration Date	Used ?

Piezoresistive Accelerometers

Sensor Number	Sensor Calibration	Calibration Date	Used ?
496	365	6-7-01	✓
497	370	6-7-01	✓
552	330	10-16-01	✓
2037	350	10-19-01	✓