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Chimney Foundation on Drilled Piers

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SYNOPSIS This paper describes the design and compares predicted performance to actual responses of a drilled pier foundation supporting a 305m high chimney. The purpose was to evaluate laboratory and empirical side friction and end bearing criteria used in the pier design. Based on results of a subsurface exploration program, and consideration of vibration effects on nearby structures, a foundation system was designed consisting of 38 drilled piers capped with a concrete mat. The piers had an average diameter of 1.37m in soil and 1.22m in rock. The average length of pier was 15.63m including a rock socket 2.44m deep. Each pier was designed to support a maximum compressional load of 1,362 tons. The side friction and end bearing capacity was analyzed from data accumulated under construction and service conditions. A comparison of this analysis with criteria suggested by others indicated compliance with accepted design standards.

INTRODUCTION

Bedrock at depths varying from 7.3m to 11.6m below ground surface in the chimney area consists of successions of micaceous sandstones with shale laminations and occasional layers of mudstone. The upper sandstones are fine-grained but grade coarser with depth. These sandstones are of Pennsylvania age and belong to the Allegheny formation. The natural overburden soils are irregular in composition, but generally consist of unsorted sand/gravel mixtures with variable amounts of cobbles, boulders, silt and clay.

The chimney foundation was constructed on piers designed based on results of a rock testing program, visual inspection of 54.9m of rock-core from the chimney foundation area, and comparison and empirical correlation of actual case histories. Load tests were not performed, as they are uncommon and generally uneconomical for high capacity piers (> 908 tons). Performance of the pier foundation was monitored during and after the chimney construction. Results concluded that the design of the pier foundation meets generally accepted design standards.

GEOTECHNICAL INVESTIGATIONS

The site condition was determined from eight engineering borings and two rock probes. The borings, drilled and sampled at the site in 1980, varied in depth from 6.2m to 31.7m. Four were redrilled to extend 4.6m to 15.2m into the sandstone bedrock. The two rock probes were drilled into rock 3.05m deep without sampling. A cross hole seismic survey was performed to evaluate the dynamic properties of the overburden soil.

A number of soil samples extracted from the borings were subjected to laboratory testing.

Soil tests included classification tests. Resistivity and pH tests were performed to assess corrosion potential. When low pH values were encountered, tests for sulphate content were performed to assess the need for sulphate resistant cement in concrete construction. Selected rock cores were tested in unconfined compression to assess rock strength. Density tests were performed for correlation to strength tests. The pier-rock skin friction and rock bearing capacity used in the design were based on the results of the unconfined compression and density tests.

SUBSURFACE CONDITIONS

The chimney is one of the new structures at the power plant, which is located adjacent to the Allegheny River. (See Figure 1.) Most of the existing structures surrounding the chimney are founded in natural soils on spread footings and mat foundations at or below elevation 251m, NGVD. The plant grade is generally at 254.5m. All these foundations were designed for bearing values around .29MPa.

The pH and sulphate content tests indicated that the natural soils in the construction areas possess pH values around 4.1 ± 0.2 throughout the soil profile down to the water table and that the pH values around 4.1 are associated with SO_4 concentrations in excess of 1,000 parts per million (ppm). The ground water table was found at 6.4m below the surface.

Top of bedrock is encountered at elevation 228.6m adjacent to the Allegheny River, and rises gently to the west up to elevation 239.3m over a distance of about 198.1m. A subsurface profile at the chimney location is shown in Figure 2.



Figure 1. General View of a 304.8m Chimney.

The entire rock-column encountered in this investigation comprises sandstones of variable composition. It exhibits fine-grained to coarse-grained with depth. Good correlation of the sandstone types was obtained by visual examination, and confirmed by density tests. For convenience of description, the rock-units have been noted as A, B, C and D in Figure 2.

Sandstone A is a weathered and fractured fine-grained sandstone. The Rock Quality Designation (RQD) of this rock is 0 percent.

Sandstone B is a dark gray, fine-grained thinly bedded micaceous sandstone with occasional shale laminations. The horizontal fracture frequency is about 1 to 4 fractures per 30 cm with predominant core lengths in the region of 12.7-15.2 cm. The RQD ranged between 50 and 90 percent. The average total density of this sandstone was 2.64g/cm^3 with a standard deviation of $.024\text{g/cm}^3$ from 43 tests. Six unconfined compression tests were performed on fresh rock cores obtained between elevations 241.7m and 234.4m. The unconfined compressive strength ranged from 165.6MPa at elevation 241.7m to 69MPa at elevation 234.4m. The bedding planes Sandstone B is thus classified in the range of high to medium strength.

Sandstone C is a fine to medium-grained sandstone grading coarser with depth. It is frequently bedded with bedding planes varying

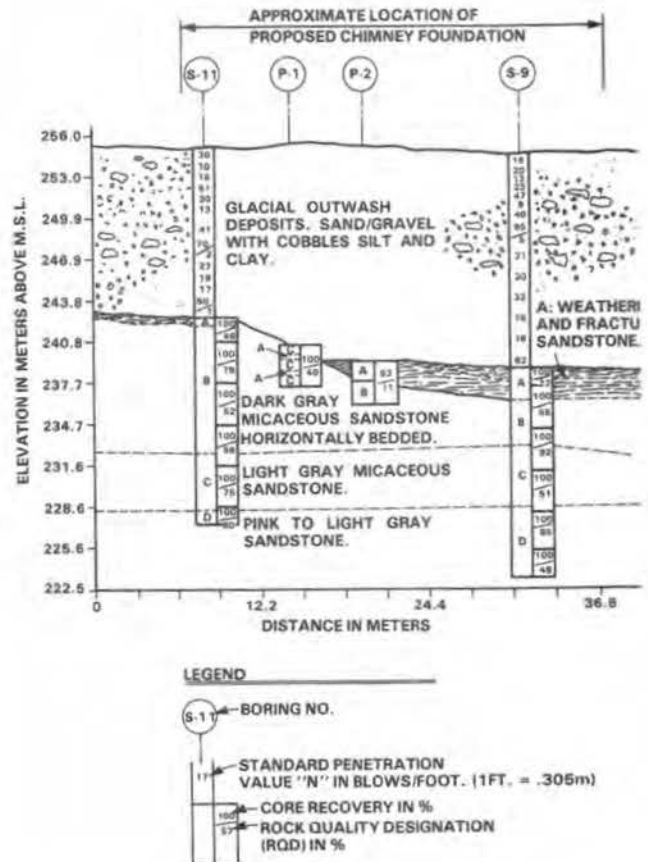


Figure 2. Subsurface Profile at Location of Proposed Chimney.

from horizontal to about 30° below elevation 232.3m. It becomes light gray and massive. The density decreases with depth and generally ranges from 2.5 to 2.6g/cm^3 at the top of the stratum to 2.36 to 2.48g/cm^3 at the bottom layer at elevation 228.6m. It is estimated that the uniaxial compressive strength of these sandstones is in the range of 41.4 to 69MPa. The RQD of this stratum ranges from 5 to 90 percent, and the rock core pieces range in length from 7.6 to 61cm.

Sandstone D is a light gray to pink generally coarse-grained sandstone. It is massive bedded with occasional lenses of dark gray mudstone. Current bedding is frequently observed, and the bedding orientation varies from 0 to 40° with the horizontal. The rock is well cemented and hard.

DESIGN CRITERIA

The dead load transmitted to the rock socket by the chimney, the foundation and piers, and backfill soils is estimated to be about 63 tons per pier. The design wind load based on a maximum wind velocity of 145km/hr translates into a maximum vertical load of 726 tons per pier for the outside ring of piers. The estimated net effective weight of the shortest pier is about 31.8 tons.

The estimated extreme and maximum operating loads that may be transmitted to the rock sockets are summarized below:

1. Maximum compressional load per pier = 1,362 tons
2. Maximum tensional load per pier = 90.7 tons
3. Maximum static load per pier = 635 tons

The drilled piers were rock-socketed into the competent Sandstone B and designed to develop the full pier capacity from side shear and end bearing in this rock socket. Likewise, all uplift is to be derived from rock socket side-shear. The overburden soils and weathered Sandstone A would contribute insignificantly to both bearing and uplift capacity and were not considered in design.

The side shear and end-bearing values used for design were developed based on the rock testing program, visual inspection, and classification of 54.9m of rock core from the chimney foundation area, and the comparison and empirical correlation of actual load tests (GAI, 1979; Horvath, 1978; Koutsoftas, 1981; Reese et al., 1977; Rosenbert et al., 1976; Winterkorn et al., 1975). Tables I and II summarize the methods used to evaluate side friction and end-bearing in rock, respectively.

Based on the unconfined compressive strength of 69MPa, and the 28MPa concrete strength for the pier, a side shear value of .7MPa was selected for design. This value provides a factor of safety of at least 1.5. The end-bearing value for the socket on Sandstone B was selected to be 7.2MPa for design. Side shear values in tension were reduced in relation to compressive values due to volume change behavior and the horizontal fracture system in the rock mass. For uplift design, the resistance of the overburden soils was neglected and the tensile side friction resistance was reduced to 60 percent of the compressive side friction.

The chimney foundation relies on passive soil resistance and the lateral capacity of the individual piers for lateral restraint of the horizontal forces. The estimated base shear was determined to be 1089 tons. The total available lateral resistance is about 2723 tons, providing a factor of safety of 2.5 against lateral movement and instability.

PIER INSTALLATION

Installation of each pier was initiated by augering a shallow oversized hole through the surface. When water was reached, a 1.52m diameter casing approximately 7.9m long was installed below the ground water table by a vibratory driver. The casing was then cleaned and the hole advanced with a 1.32m diameter auger. Weathered rock was reached at 11.6m in Pier 1 and at 7.3m in Pier 12. The auger was advanced to refusal in the weathered rock. A 1.32m casing was then set and drilled into

Table I. Evaluation of Side Friction (fb) Between Rock Socket and Rock.

Rock Type	Correlations with UC (MPa)	By Actual Tests (MPa)	Remarks
General ¹	fb = 4/UC fb = 0.05 (UC)		Based on load tests Based on strength ratio
General ²	2.45 to 3.2 for UC = 70.3		
Sandstone, with shale, highly fractured, unevenly weathered, extremely variable strengths ³		1.0	Pier compression test D = 51cm L = 2.59m L/D = 5 f'c = 35MPa End Bearing = .49MPa
Sandstone, weakly cemented with some shale layers ⁴	UC = 12 to 24.5	.91	Plug compression test D = 24.5cm L = 64cm L/D = 2.6 Tremie concrete f'c = 38MPa
	UC = 12 to 24.5	.84	D = 24.5cm L = 76cm L/D = 3.1
	UC = 12 to 24.5	1.27	D = 24.5cm L = 94cm L/D = 3.9
General ⁵	fb = 0.05 UC < 1.4	1.03 to 2.7	
General ⁶	1.4 fb = 0.03 to 0.05 UC (from .84 to 1.4)		
Siltstone ⁷	.31 for UC = 42.9		
Sandstone ⁸		1.75	Anchor pull-out test D = 15.2cm; L = 91cm L/D = 6 f'c = 42MPa Bond = .32MPa used for design
Sandstone and siltstone, alternating zones UC = 7MPa (mean value) ⁹		.53	Anchor pull-out test D = 10.2cm L/D = 90 Anchor holes reamed and flushed clean, pressure grouted

Legend: UC = Unconfined compressive strength in MPa
L = Length of pier socket
D = Diameter of pier
f'c = Ultimate compressive strength of concrete

Notes: ¹Horvath, et al., 1978
²Rosenberg and Journeaux, 1976
³Moore, 1964
⁴Vogan, 1977
⁵Woodward, et al., 1972
⁶Various building codes (New York City, Others)
⁷GAI Consultants, Inc., 1979
⁸Drossel, 1970
⁹Moss, 1971

Table II. End Bearing Evaluation (qa) of Values.

References	Presumptive Allowable (MPa)	Actual Tests (MPa)	Remarks
1. Various building codes	1.46 to 9.7		
2. Uniform building code	1/5 UC		
3. Reese, 1977	1/5 to 1/10 UC		
4. D'Appolonia, 1975 (see Winkerkorn, et al., 1975)	1/5 to 1/8 UC		
5. GAI, 1978		6.3 (UC = 52.7MPa)	For hard siltstone

Sandstone A. The rock socket was drilled with a 1.22m diameter core barrel. Because a pre-glacial cliff edge or rock-slope in the

western section of the area was located from the geotechnical investigation, all rock sockets had to be drilled into Sandstone B to a minimum depth of 2.44m. All rock sockets were inspected subsequent to drilling and dewatering by an inspector. Plumbness was checked for each pier and ranged from 0.6 to 1.2 percent.

After socket inspection, a reinforcing cage, consisting of 16 - No. 11, Grade 60 420 MPa reinforcing steel bars, was set in the pier. The 1.52m casing was removed and the oversized hole was backfilled and tampered with clean sand. Accumulated water inside the socket was pumped out and concrete ($f'c = 28$ MPa) was placed through a hopper and pipe well into the casing prior to the 1.32m casing being removed. Additional concrete was added to reach grade.

PIER INSTRUMENTATION

The performance of the pier foundation was monitored during and after the chimney construction with 24 strain gauges. The gauges were installed in pairs at six different elevations in Piers 1 and 12. The instrumented piers were diametrically opposite each other in the foundation. The strain gauges used were Micro-Measurements Gauge Series CEA and were attached to the rebar in the reinforcing cage. The leads were run up the reinforcing steel and strung through conduit from the top gauge to near the top of the dowel. The strain gauge data from 24 gauges were obtained manually by means of a strain indicator during and after chimney construction.

RESULTS AND INTERPRETATION

The behavior of Piers 1 and 12 was monitored at the following stages: Initial reading on May 9, 1981, after completion of Piers; Second reading on August 6, 1981, after completion of pier cap; Third reading on August 23, 1981, after chimney construction to 48.8m; Fourth reading on September 20, 1981, after chimney construction to 192m; Fifth reading on December 28, 1982, after chimney construction to 305m and in service. The pier load history is shown in Figure 3. The wind velocity during the last two sets of readings was estimated to be less than 32 Km/Hr which would increase or decrease pier strain at a magnitude of less than 20×10^{-6} . This strain change is equal to about 10 percent of the elastic strain.

During construction, the instrumentation boxes were relocated from positions where the initial readings for Pier 1 and 12 were taken, thus invalidating these readings. Therefore, the second reading was treated as the initial reading. Furthermore, the strain data from Pier 12 and from the fifth reading in Pier 1 could not be correlated and were not considered representative as temperature changes in the wires affected the accuracy of the gauge readings. The measured load transfer in Pier 1 and the estimated load transfer curves as a function of depth under different loading stages are shown in Figure 3. Due to the high

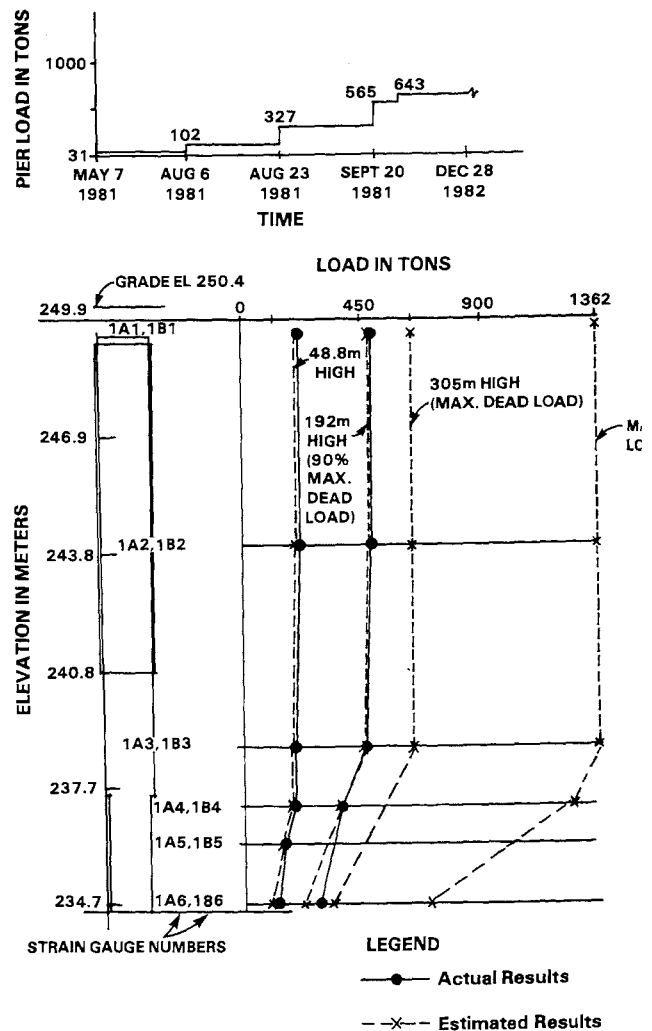


Figure 3. Load in Pier No. 1 as a Function of Depth.

capacity of the pier, the medium dense overburden soils contribute only insignificant frictional resistance.

The measured end bearing resistance at each stage is larger than the frictional resistance and is approximately 12 to 20 percent higher than predicted values. The frictional resistance of the rock socket is not fully mobilized at the low stress level within the elastic limit causing this difference. However, all measured strains were within the computed elastic limit.

CONCLUSIONS

Based on construction and in-service performance of these large diameter, high capacity, drilled piers, the following conclusions were drawn:

1. Field results (corrected for temperature and wind) of the load transfer to the rock socket compares reasonably, up to 90

percent of the maximum dead load, with predicted results.

2. The evaluation of side friction and end bearing between the rock and the pier based on unconfined compressive strength and its RQD provides reasonable results. A careful inspection of the rock core and study of similar case histories is recommended in selection of design values.

3. Full scale load tests on high capacity drilled-piers (> 908 tons) are not common and are generally uneconomical. It is recommended that instrumented small scale load tests be performed at different stress-strain levels on the affected foundation rock.

4. The outer ring piers were designed to resist a maximum compressive wind load of 726 tons each (145 Km/Hr). Instrumentation to measure wind loading is recommended to accurately assess the stress at varying strain levels.

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