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ABSTRACT

A very large mobile crane was used to lift a 3,150 kN steam generator through the roof of the containment building of a nuclear power plant. The maximum load on the crane was 19.8 MN, giving a track pressure of almost 600 kPa. Soil conditions were stiff clay underlain by softer clay. This paper describes the bearing capacity and settlement analysis performed to establish a suitable shallow foundation for the crane. The foundation load test confirmed that soil conditions had been adequately defined and that the foundation design was satisfactory.

KEYWORDS

Bearing capacity; load test; mat foundation; piezocone; settlement; soft clay.

INTRODUCTION

Two 3,150 kN steam generators were lifted by crane through the roof of the containment building (approximately 35 m high) during steam generator replacement at Rochester Gas and Electric's Ginna Nuclear Plant and were replaced with two new generators. The calculated maximum crane vertical loading was about 19.8 MN, equivalent to a bearing pressure of 575 kPa on each of the two crane crawler tracks. Borings made in the crane operating area before the lift indicated a 3- to 4-m thick layer of stiff clay underlain by a 4- to 5-m layer of soft to medium stiff clay, and then hard silt and bedrock. Given the extent of the soft clay, and the critical requirement for foundation stability, driven or drilled piles were considered for crane foundation support. However, since the fully loaded crane would have to maneuver extensively during rigging of the steam generators, the estimated number of piles was substantial, as was the cost (on the order of $500,000). Detailed piezocone and field shear vane investigations were conducted to provide additional data to explore alternative solutions to piled foundations. The results indicated that the crane could be supported on an appropriately proportioned soil-supported mat foundation.

This paper summarizes the exploration program and then focuses on the foundation mat analysis and design, construction, and load testing.

CRANE DESCRIPTION

The steam generators were lifted with a Lampson LTL-1200 Transi-Lift Series 2A (Fig. 1), one of the world's largest mobile cranes, consisting of two crawler units separated by a 30 m stinger (the rear unit being the counterweight component). Each crawler unit was independently powered and could move independently of the other, to the extent allowed by the stinger. The design of the boom, mast, and swivel on the front crawler resulted in equal pressure along the length of the tracks, i.e., no eccentric loading. The counterweight load on the rear crawler was also equally distributed on the tracks.

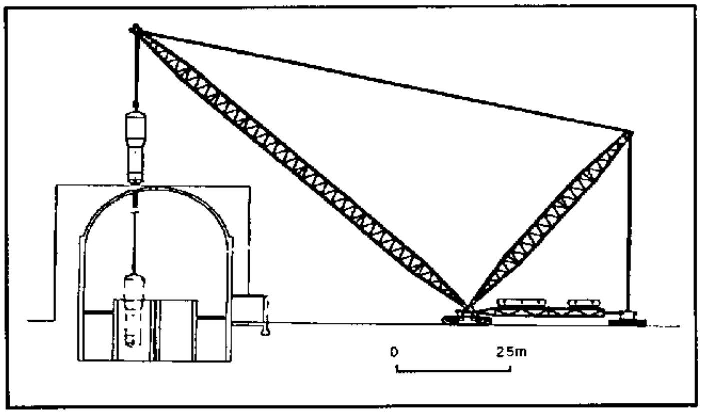

Fig. 1 Transi-Lift Lifting Steam Generator from Containment Building

The maximum loading conditions on the front and rear crawlers depended on the lifted load, the working radius, and the amount of counterweight. Wind loading and impact were included. The maximum working radius at steam generator lift was about 75 m. The maximum front crawler track reaction during test load and maximum lift was 19.83 MN, giving an average pressure of 575 kPa on each of the 1.83 m x 9.44 m tracks. The maximum rear crawler track reaction load occurred under full counterweight load, with no hook load and the boom at maximum angle to the horizontal. This reaction was about 16.03 MN, giving an average bearing pressure of 528 kPa on each of the 1.83 m x 8.31 m tracks.

SUBSURFACE INVESTIGATIONS AND CONDITIONS

Figure 2 shows subsurface conditions interpreted from sample borings. The main plant structures are founded on the sandstone of the Queenston formation or on the thin layer of overlying hard till (Layer 3). The layers of interest for Transi-Lift support were the Layer 1 stiff to very stiff silty clay, and the Layer 2 soft to stiff silty clay. According to Rochester Gas and Electric Corporation [1994], these clays are glacial and lakebed deposits. The Layer 1 clay is a till deposit, while the underlying softer Layer 2 clay is a lakebed material believed to have been deposited in the bed of a former glacial lake, Lake Iroquois. Groundwater is typically at about mid level in Layer 2.

Although the Layer 1 clay showed a considerable variation in SPT N-value (Fig. 2), it was a significantly overconsolidated deposit, with average moisture content close to the plastic limit. Various tests were made on the Layer 2 clay to estimate its undrained shear strength (S_u) value, including unconsolidated undrained triaxial, torvane, and pocket penetrometer tests, and S_u versus N-value correlations. There was considerable variation in the strength estimates, although there was no significant trend with depth. The results suggested that Layer 2 was somewhat overconsolidated. Given the scatter of the Layer 2 strength results, and the high cost of constructing a deep foundation system, the decision was made to conduct a detailed piezocone and field shear vane program to confirm (or not) that there was adequate Layer 2 shear strength available to support a mat foundation for the Transi-Lift.

Fifteen piezocone soundings at approximately 12 m spacing were conducted, along with a porepressure dissipation test and a seismic cone penetrometer test. Field vane shear tests were performed at six of the piezocone locations, at an average of three depths per location. The details and results of these tests are presented in Davie *et al.* [1998]. The derivation of soil shear strength and elastic and consolidation properties of the clays made in Davie *et al.* [1998] is summarized below.

FOUNDATION ANALYSES

Of primary importance was the derivation of a lower bound undrained shear strength for the Layer 1 and Layer 2 clays to confirm the adequacy of the bearing capacity of a shallow mat foundation. Undrained shear strengths derived from the piezocone and shear vane tests were extremely consistent and constant with increasing depth in Layer 2, and showed reasonable agreement with the triaxial test results. The piezocone results indicated a transition zone of about 0.75 m thickness between Layer 1 and Layer 2. For bearing capacity design purposes, Davie *et al.* [1998] indicates S_u = 50 kPa for Layer 2; for Layer 1, S_u =145 kPa, with linearly decreasing values in the transition zone between the bottom of Layer 1 and the top of Layer 2.

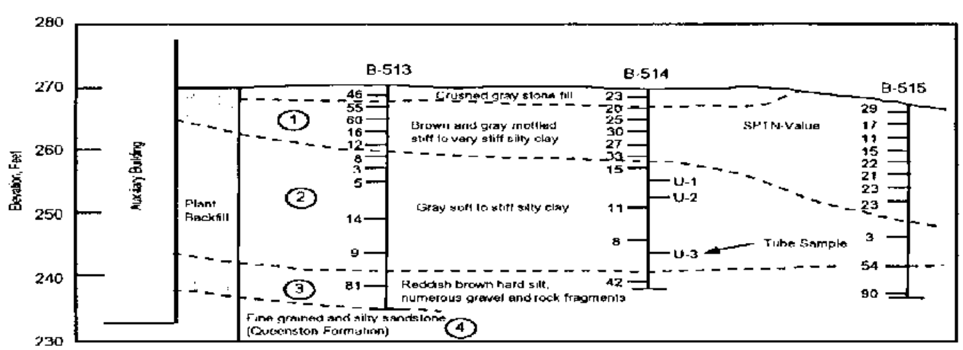

Fig. 2 Subsurface Profile in Crane Operating Area

Estimation of total and differential settlement and rate of settlement were also considered necessary for the safety of the loaded crane, since the manufacturer limited the differential settlement between the tracks to 20 mm. Davie *et al.* [1998] indicates that, using empirical relationships between S_u and modulus of elasticity (E), E values of 87 MPa and 30 MPa resulted for Layer 1 and Layer 2, respectively. The ratio of this Layer 2 E value to the low strain E value derived from the seismic cone penetrometer tests was almost 14. According to Sun *et al.* [1988], this ratio in clays would typically occur at strains of about 3 percent, compared to the 0.1 percent to 0.2 percent elastic strain anticipated. This result tended to confirm the lower bound assumption of S_u for Layer 2. The values of compression ratio and recompression ratio computed from Layer 2 consolidation tests were 0.143 and 0.0062, respectively. These values were also used for Layer 1. The vertical coefficient of consolidation C_v from consolidation tests ranged from about 19 to 32 mm^2/sec for the maximum load. The average horizontal coefficient of consolidation C_h from the piezocone porepressure dissipation test was 22 mm^2/sec. Although C_h is typically greater than C_v, the agreement was considered reasonable.

Allowable Bearing Capacity

For soil bearing capacity analysis, a minimum factor of safety (FS) of 3 is typically used, mainly to take into account variations in soil strength and loading and to allow for approximations in the analysis. In the crane operating area, the thorough subsurface investigation provided a relatively high level of certainty for the soil layer strength and thickness, so a reduced FS might usually have been considered. However, the FS chosen also reflects bearing failure consequences which, for the crane lifting the steam generator out through the roof of the containment building, would have been extreme. Also, the nature of the crane loading was more critical than in a normal structure situation, where the loading increases gradually over a long period; in the Transi-Lift case, the load would be applied over a very short period, allowing no stress/strain adjustments in the subgrade. Thus, a minimum FS of 3 against soil bearing failure was considered necessary, averaged over the area of the foundation.

Computations indicated that, for the crawler tracks bearing directly on the Layer 1 clay, the FS against bearing failure under maximum load computed using Skempton [1951] was about 1.1. It was apparent that a fairly rigid foundation that could distribute the loads into the soil was needed. This foundation had not only to take into consideration potential failure in the relatively stiff Layer 1 clay, but also failure in the weaker Layer 2 clay, either by punching through the upper clay into the lower clay, or squeezing out the lower clay. Although a larger foundation area supporting the crane would result in a lower average pressure transmitted to Layer 1, there would be less attenuation of pressure with depth, and the Layer 2 clay would become critical. Also, since the stiffer clay was on top, a foundation placed on the ground surface would transmit less stress to the weaker lower clay. However, such a foundation would have to be demolished after steam generator replacement. The selected foundation was flush with the ground surface.

The bearing capacity analysis for the fully loaded front crawler was made assuming a 1.15-m thick highly reinforced concrete mat that extended approximately 2 m beyond the outside edges of the crawler tracks, resulting in mat dimensions of 15.85 m x 14.63 m. The method used was based on Brown and Meyerhof's approach (Brown and Meyerhof [1969]) as modified by Vesic (Vesic [1975]) for punching failure in layered soils. For squeezing of the Layer 2 clay, the stress at the Layer 2 surface was computed, and Skempton's approach (Skempton [1951]) was used. FS was calculated for each of the 15 piezocone-derived strength profiles. Assuming the mat distributed the total track pressures evenly into the soil, the minimum punching FS at any of the locations was 3.75, with an average of 4.1. The corresponding FS values against squeezing of the Layer 2 clays were 4.3 and 4.85.

The assumption that a 1.15-m thick concrete mat that extends 2 m beyond the edges of the crawler tracks would distribute the track loads evenly to the subgrade was an approximation, since there would inevitably be stress concentrations beneath the track footprints. The stress distribution across the mat was estimated using a finite element analysis, with the supporting soil modeled using spring values based on the results of the elastic settlement analysis (see below) on the fully loaded mat. The analysis used a cracked section that reduced the effective thickness (stiffness) of the mat, reducing its stress distribution capabilities. Also, the tracks analyzed were offset from the centerline of the mat by the maximum offset tolerance of 250 mm. As anticipated, the maximum computed pressures occurred beneath the tracks, and on a section perpendicular to the tracks through the center of the crane. The highest single element pressure obtained was about twice the average pressure applied to the mat. The effects of such a pressure distribution were assessed considering: (1) the type of failure that provides the minimum FS (shearing through Layer 1 into Layer 2) cannot occur under locally higher stressed areas of the mat without shearing the mat itself; (2) the average FS against a bearing failure of the mat is 4.1, regardless of how well or poorly the stresses are distributed; and (3) the thickness of the lower clay is only about one third of the width of the foundation mat, and thus will limit failure surfaces from developing in Layer 2.

Based on the above, it was concluded that the 1.15-m thick mat would be a satisfactory foundation for the front crawler. A similar analysis was conducted for the rear crawler; because of the somewhat smaller loading, a 1-m thick mat was found to be satisfactory.

Computed Settlements

The average computed settlement of the fully loaded mat due to elastic compression of the clay layers was about 12 mm, and thus elastic settlement presented no problem with regard to the 20 mm limit on track elevation difference. Computations (Davie *et al.* [1998]) indicated that the Layer 2 clay would not undergo virgin consolidation, even under maximum loading. Even if the loading were to cause a virgin consolidation condition, the length of time the fully loaded front crawler would be stationary would be too short to cause significant consolidation settlement.

FOUNDATION PERFORMANCE

The front crawler foundation was load tested before the first steam generator lift, using deadloads consisting of concrete blocks and gravel-filled boxes to simulate the maximum loading on the soil. The weights were arranged so that the pressure was greatest at the crawler track locations. The maximum load height at the track locations was about 8.5 m. Settlements were measured optically on 6 points around the perimeter of the mat; each point was within 0.3 m of the edge of the load stack. The results are summarized on Table 1. The maximum recorded settlement was about 6 mm. The average settlement under maximum load was just under 4 mm. The settlement pattern was relatively elastic, although some of the settlement under the initial 4.11 MN load was probably due to seating adjustments, as evidenced by the settlements recorded after the weights had been removed. Because of construction scheduling, the 4.11 MN load was left in place for over 5 weeks. No increase in settlement was recorded during that period.

During the steam generator lifts, settlement was measured optically at 8 points on the perimeter of the front crawler mat, and at points at the centers of the outside edges of the two crawler tracks. Recorded settlements were significantly less than during the test load. The maximum settlement recorded at full lift was less than 2 mm. No distress was observed in either of the mat foundations during the load test or the steam generator lifts.

CONCLUSIONS

The results of the detailed subsurface investigation program provided the required degree of confidence to support the crane on shallow mat foundations. The load test results confirmed that a sufficient amount of conservatism had been incorporated into the mat analysis and design.

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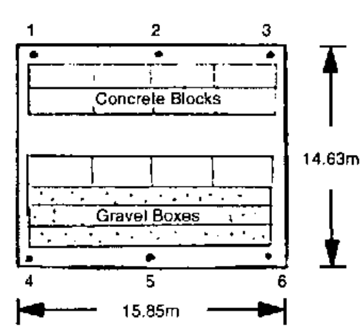
Table 1 Load Test Settlments

Point	Recorded Settlement, mm			
	4.11 MN	9.79 MN	19.83 MN	Unloaded
1	3	6	6	3
2	0	3	4.5	0
3	0	0	0	0
4	3	3	6	3
5	0	0	3	0
6	3	3	3	3
average	1.5	2.5	3.75	1.5



Front Crawler Mat Plan