PERFORMANCE OF PILE FOUNDATIONS SUBJECTED TO DOWNDRAG

by

Alan B. Wagner[1], P.E. - Fellow ASCE, and Van E. Komurka[2], P.E. - Member ASCE

Abstract

Two pile-supported terminal buildings at the Port of Milwaukee have experienced large, continuing settlements since their construction in 1961. The 47.5 m (150 ft) by 152.4 m (500 ft) buildings were constructed on 7.9 to 8.5 m (26 to 28 ft) of fill placed within a sheet pile enclosure. The buildings' columns are supported on wood piles which are 19.8 m (65 ft) long, and extend only slightly below a 10.7-m-thick (35-ft-thick) compressible organic clay deposit which settled as a result of fill and floor loads. Design floor loads equaled 28.7 kPa (600 psf), and the floors were constructed of flexible asphaltic concrete to accommodate anticipated total and differential floor settlement. In 1993, the terminal buildings were first used for storage of steel coils, which have applied a contact stress commonly as high as 129 kPa (2,700 psf), and occasionally as high as 192 kPa (4,000 psf), over wide areas. Analyses which considered pre- and post-construction subsurface profiles, pile length, and pile loads indicate that for both floor loading conditions, the piles' neutral plane was at or above the top of the organic clay deposit. Since pile settlement is approximately equal to soil settlement at the neutral plane, and since the organic clay deposit settled, large pile settlement resulted. The paper discusses measured and predicted soil settlement, neutral plane locations, and how pile performance is related to the soil settlement profile.

Project Description

Terminal Building Nos. 3 and 4 are pile-supported structures built in 1961 on South Pier No. 2 at the Port of Milwaukee, Wisconsin. Each building measures 45.7 m (150 ft) (north-south) by 152.4 m (500 ft) (east-west), and was built on fill placed to construct the pier. This fill was placed within a sheet pile enclosure which extends 311.2 m (1021 ft) eastward into Lake Michigan. The transverse (north-south) dimension of the pier is 158.4 m (520 ft). The pier's sheet pile walls are located under the inside edges of pile-supported relieving platforms along the longitudinal (east-west) sides of the pier. The sheet pile walls are anchored by 1 level of H-section struts which extend back to the nearest interior pile cap within the buildings, and to pile-supported A-frames between column locations. Each building is a 3-span structure in its transverse direction, with each outer span measuring 13.7 m (45 ft), and the center span measuring 18.3 m (60 ft). The lakeside column row of each building is integral with the pile-supported relieving platform. The 4 columns in the building's transverse direction support roof trusses located 6.4 m (21 ft) on center. The dead load from the building superstructure on each column is on the order of 534 kN (60 tons); the live load on each column is negligible.

[1] President, Wagner Komurka Geotechnical Group, Inc. - Milwaukee, Wisconsin

[2] Vice President, Wagner Komurka Geotechnical Group, Inc. - Milwaukee, Wisconsin

At the time of construction, the lakebed was nominally 4.9 m (16 ft) below lake level (lake level = Elevation 0). Design floor grade ranges from Elevation 3.0 to 3.7 m (10 to 12 ft), requiring fill thicknesses on the order of 7.9 to 8.5 m (26 to 28 ft). Interior building columns are typically supported on 4 wood piles, driven when the fill was slightly above lake level. The piles have a cut-off elevation of 0.2 m (0.5 ft), and a length of 19.8 m (65 ft), resulting in a typical toe elevation of -19.6 (-64.5 ft). Since the lakeward interior pile groups of each building contain inclined piles to resist the lateral loads imposed by the sheet pile walls' tie rods, typical toe elevations of the inclined piles are somewhat higher than those of the other (vertical) piles. Pile caps measuring 1.7 m (5.5 ft) square and 0.8 m (2.5 ft) thick were constructed at a subgrade elevation of 0, and columns (plinths) were extended from the caps up to the floor elevation of the buildings.

To accommodate the anticipated total and differential settlement resulting from fill placement, and from design floor loads of 28.7 kPa (600 psf), the buildings' floors consist of flexible asphaltic concrete. Until 1993, the buildings' floor loads were generally limited to this design load (the initial loading condition). In 1993, in response to market changes, the buildings began being used to store stacked steel coils. The steel coils commonly imposed floor loads of 129 kPa (2,700 psf) (the final loading condition), and occasionally as high as 192 kPa (4,000 psf), over wide areas of the buildings. More-detailed documentation of the magnitudes, locations, and durations of floor loads is not available.

Subsurface Conditions

The 4 major strata comprising the typical post-construction subsurface profile are presented in Table 1.

TABLE 1

Summary of Subsurface Profile

		Elevation			
		meters		feet	
Strata No.	Description	Top	Bottom	Top	Bottom
1	Sand Fill	3.4	-4.9	11	-16
2	Native Sand	-4.9	-7.3	-16	-24
3	Organic Clay	-7.3	-18.0	-24	-59
4	Sand	-18.0	-21.3*	-59	-70*

* Maximum depth explored.

The sand fill and native sand are typically in a loose to medium dense condition. The organic clay is typically in a medium stiff to stiff condition, and has water contents typically ranging from 40 to 60 percent. Consolidation tests performed on the organic clay indicate that the deposit is overconsolidated. The relationship between the organic clay deposit's preconsolidation stress, its pre-construction effective vertical stress, and its effective vertical stress for the initial and final loading conditions is shown in Figure 1. The organic deposit's

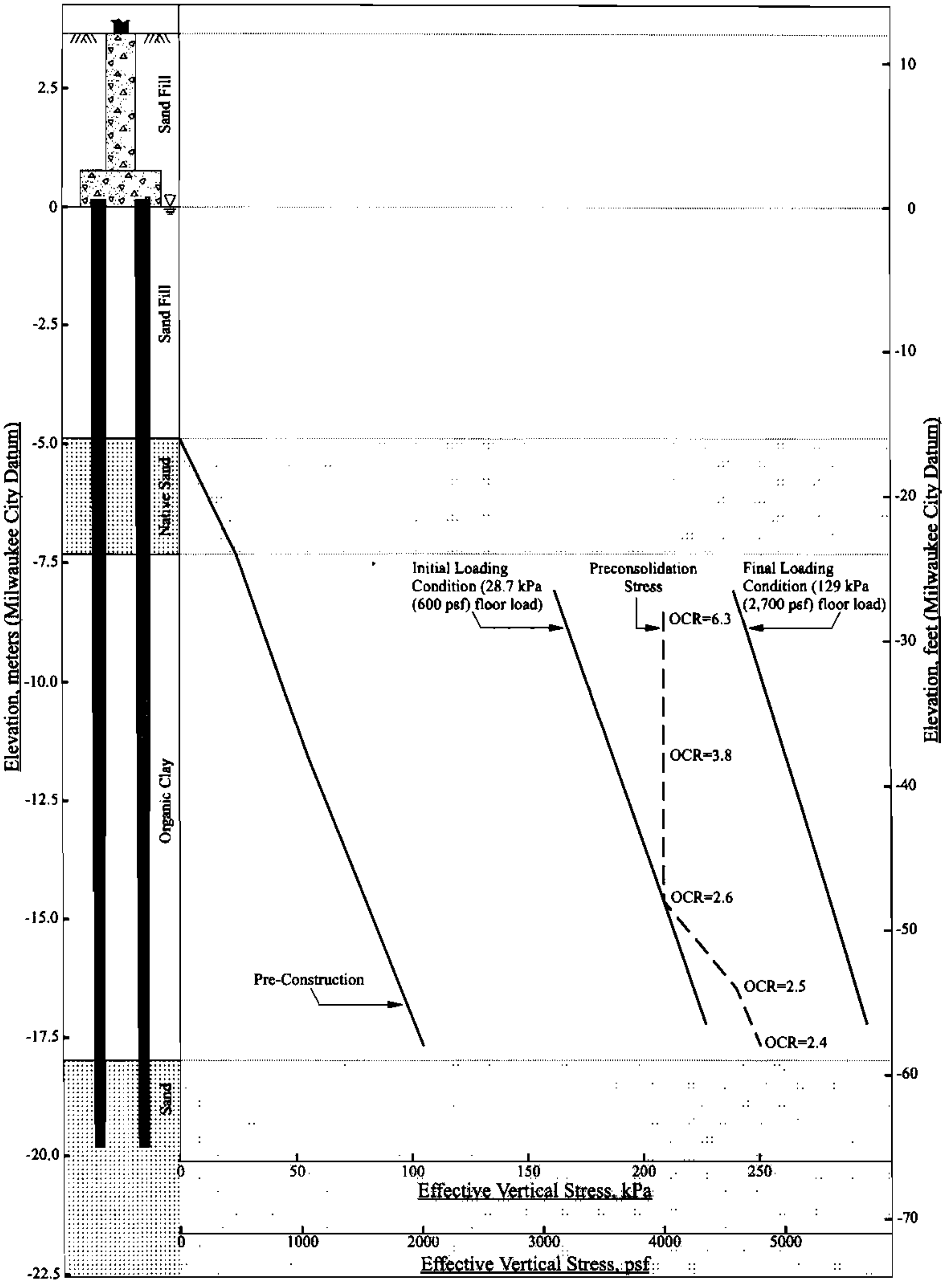

Figure 1. Effective Vertical Stresses

overconsolidation ratio (OCR) ranged from 6.3 at the top of the deposit, to 2.4 at the bottom of the deposit. A review of Figure 1 indicates that the final loading condition increased the vertical effective stress in the organic clay to beyond its preconsolidation stress for the full depth of the deposit. The organic deposit's average compression index (C_c) was 0.46, while the average recompression index (C_{cr}) was an order of magnitude lower (0.046). The sand below the organic clay was in a loose to medium dense condition.

Building Performance

The buildings' columns and floors began settling immediately upon their completion in 1961. The elevations of several columns in each building were monitored on an annual to biannual basis from 1961 to 1973. The interior columns and floors settled more than those around the perimeter. A plot of settlement versus time of a representative column near the center of each building is presented in Figure 2a. The same settlement data are presented as a function of log time in Figure 2b. These 2 columns settled 142 and 175 mm (5.6 and 6.9 in) during the monitoring period. Shortly after the monitoring period, the buildings' column-to-truss connections were repaired. During the repair program, the survey marks were inadvertently lost. However, settlement of the columns continued, and the roof trusses were damaged by differential settlement. The columns were leveled, and the roof trusses were repaired, during the winter of 1996-97.

The structural engineer designing the repairs surveyed the elevations of each column-to-truss connection in 1996. By comparing the surveyed elevations to the initial design elevations, the settlement of each column was estimated. Since the connections may have been constructed at elevations slightly different than design, these estimates are considered approximate. For Building 3, it is estimated that settlement of interior columns ranged from 142 to 279 mm (5.6 to 11.0 in), with an average of 24 survey locations equal to 213 mm (8.4 in). For Building 4, it is estimated that settlement of interior columns ranged from 269 to 518 mm (10.6 to 20.4 in), with an average of 24 survey locations equal to 406 mm (16.0 in).

The buildings' floors have been overlain to re-level them on 2 occasions, with a maximum total overlay thickness on the order of 152 mm (6 in). There has been more floor settlement at the center of the buildings than at the edges, and there is still a visually perceptible sag in the middle of the floors. Although no floor elevation data was obtained, it appears that column and floor settlements have generally been of similar magnitudes.

Soil Settlement Predictions

Settlement analyses were based on the above-referenced consolidation parameters for the organic soil deposit, and published typical compressibility parameters for granular soils (both fill and native) [NAVFAC]. Estimated magnitudes of soil settlement at various elevations throughout the subsurface profile were calculated (secondary compression was ignored). For combined loading from the weight of the fill and a sustained 28.7 kPa (600 psf) floor load, the analyses predict a floor settlement at the center of the buildings of 201 mm (7.9 in) (compared to 142 and 175 mm (5.6 and 6.9 in) measured for Buildings 3 and 4, respectively).

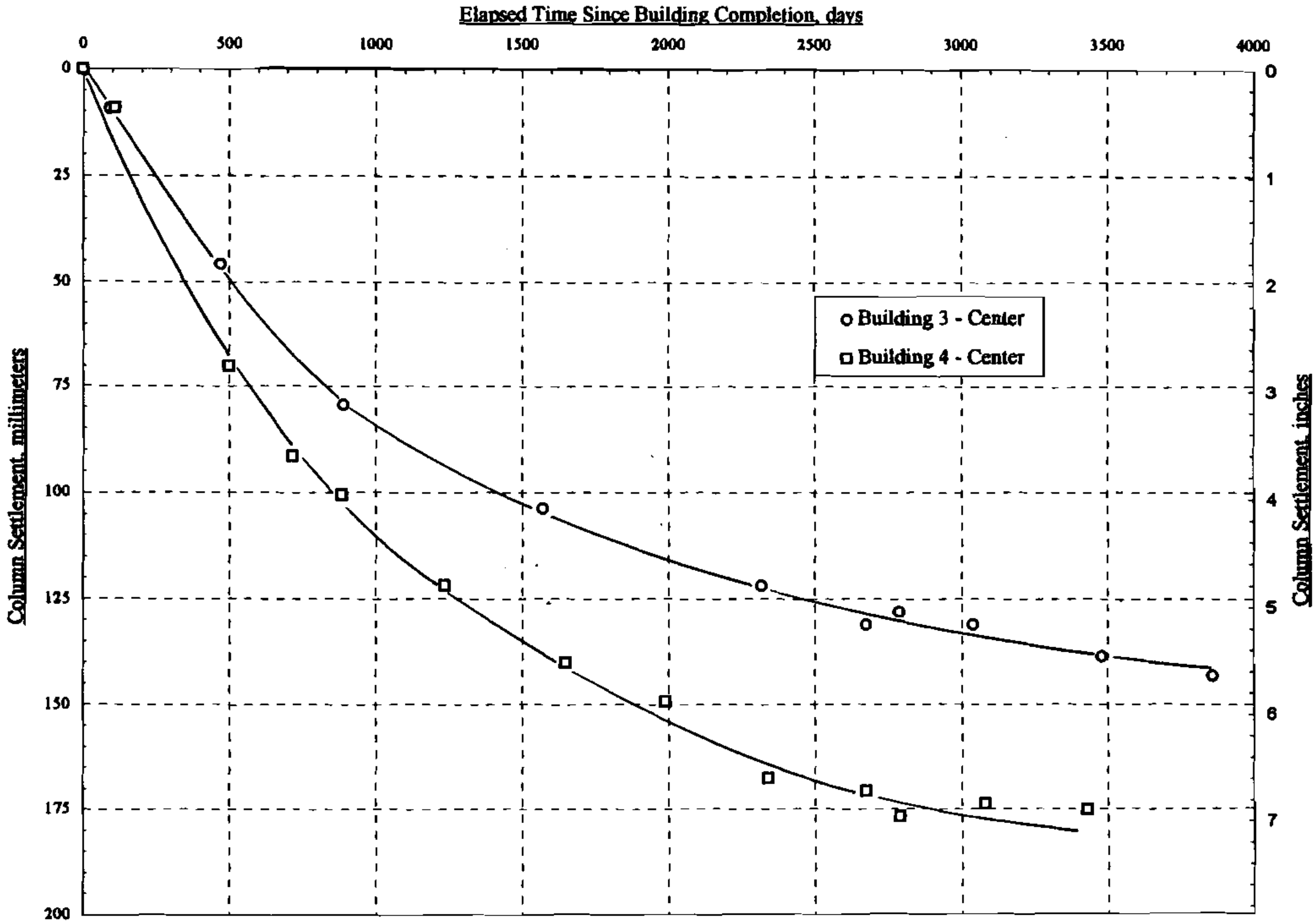

Figure 2a. Column Settlement versus Time

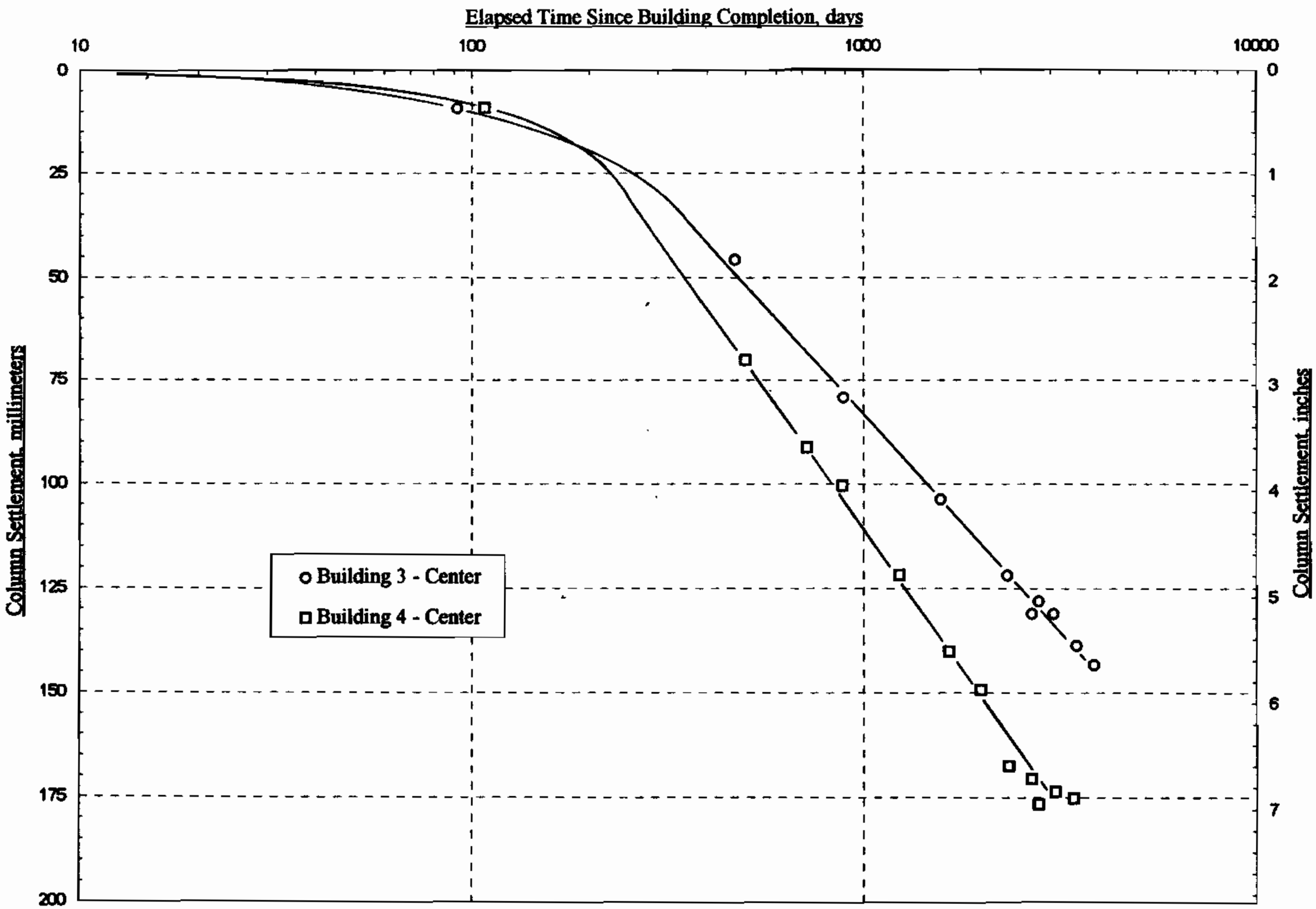

Figure 2b. Column Settlement versus Log Time

The analyses predict that as a result of floor loads increasing to 129 kPa (2700 psf) from the storage of steel coils, floor settlement at the center of the buildings would increase to 335 mm (13.2 in) (compared to an average of 213 and 406 mm (8.4 and 16.0 in) measured for Buildings 3 and 4, respectively). The results of soil settlement prediction analyses for both loading conditions are presented as plots of calculated soil movement as a function of elevation in Figure 3.

Based on the similarity of subsurface conditions encountered beneath the buildings, it appears that the difference in building settlements is largely attributable to differences in the magnitudes, locations, and durations of floor loads.

Pile Settlement Predictions

As noted above, the typical building superstructure dead load applied to each interior column was on the order of 534 kN (60 tons). This translates to a structural dead load of 133 kN (15 tons) on each of the 4 column support piles. However, this structural load does not account for: 1) the floor load above, and supported by, the pile cap (always present), 2) the weight of a soil "block" above, and supported by, the pile cap (always present), and 3) the drag friction acting on the sides of the soil block above, and supported by, the pile cap (present when the soil around the cap moves down relative to the cap).

The geometry of the of soil block above, and supported by, the pile cap is subject to a number of interpretations. For the purposes of this paper, the soil block was assumed to have vertical sides (i.e., a vertical projection of the pile cap was considered). This is likely the most-conservative assumption relative to pile loading (i.e., results in the lowest additional pile loads). Based on this soil block geometry, the dead load on each of the 4 piles would increase by 35.6 kN (4 tons) from the weight of soil, 17.8 kN (2 tons) from the 28.7 kPa (600 psf) floor load, and 53.4 kN (6 tons) from drag friction on the sides of the soil block. Including the structural dead load, for the initial floor load, this results in a total dead load of 240 kN (27 tons) per pile. The additional dead load resulting from increasing the floor load to 129 kPa (2,700 psf) would be 71 kN (8 tons) per pile. Thus, for the final floor load, the total dead load would be 311 kN (35 tons) per pile.

Distribution of subsurface forces acting on a pile was estimated using an effective-stress static pile capacity model developed from experience with numerous instrumented static load tests, largely in the Milwaukee area [Wagner]. Pile group effects were not considered. These forces include dragload (the accumulation of negative skin friction), positive shaft resistance, and toe resistance. Soil settlement resulted in the surrounding soil moving down relative to the pile (downdrag), causing negative skin friction.

Some designers assume downdrag occurs to the bottom of the deepest compressible soil deposit, and consider the full resulting dragload to always, continually act on a pile. For the subject subsurface profile, this pile loading condition is illustrated in Figure 4a. Assuming downdrag occurred to the bottom of the organic clay, analysis indicates a dragload of 329 kN (37 tons). This, added to the total dead load of 240 kN (27 tons) already applied to the pile, would result in a total load of 569 kN (64 tons). Resisting (upward) forces would include 35.6 kN (4 tons) of

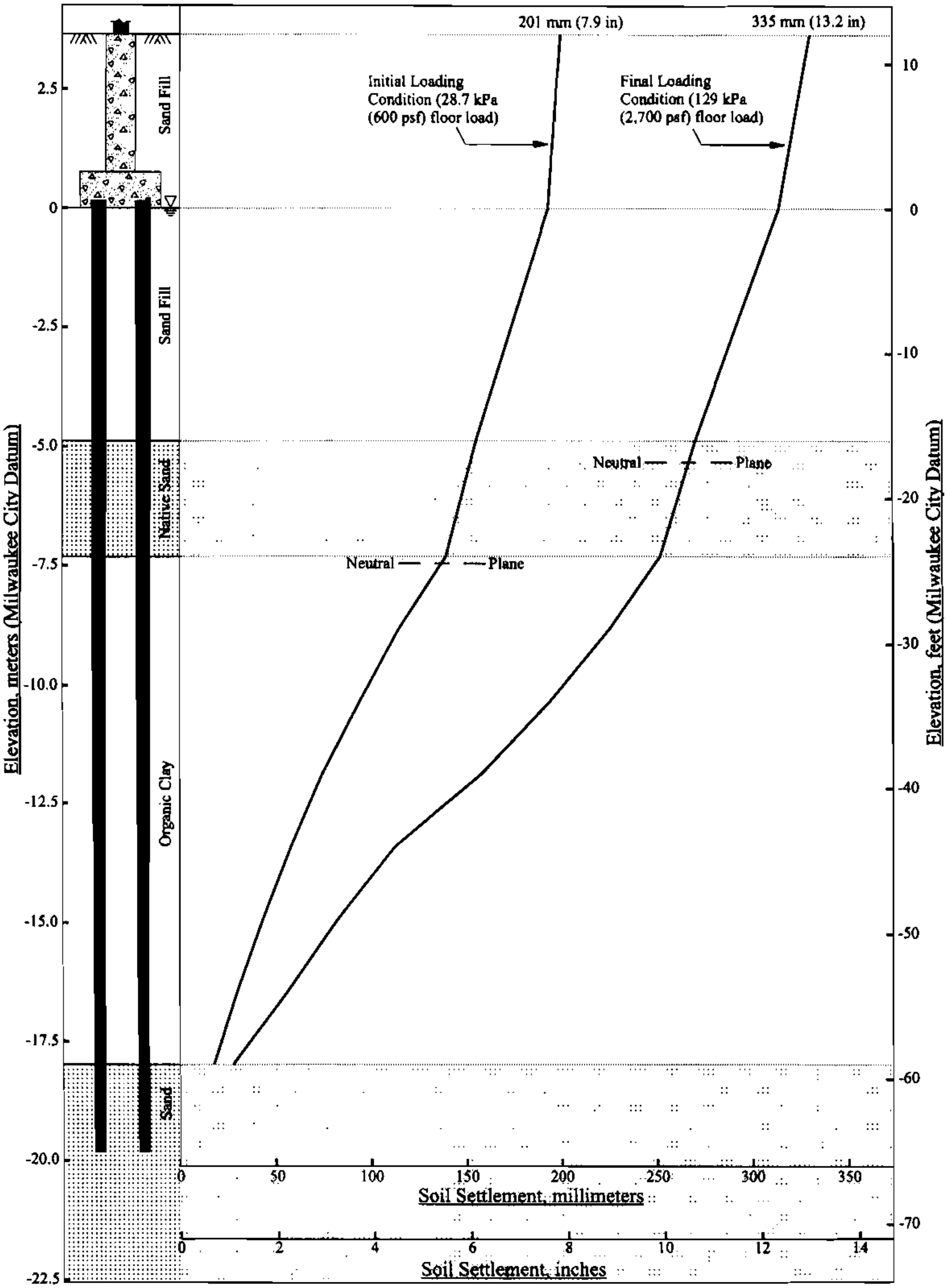

Figure 3. Calculated Soil Settlement

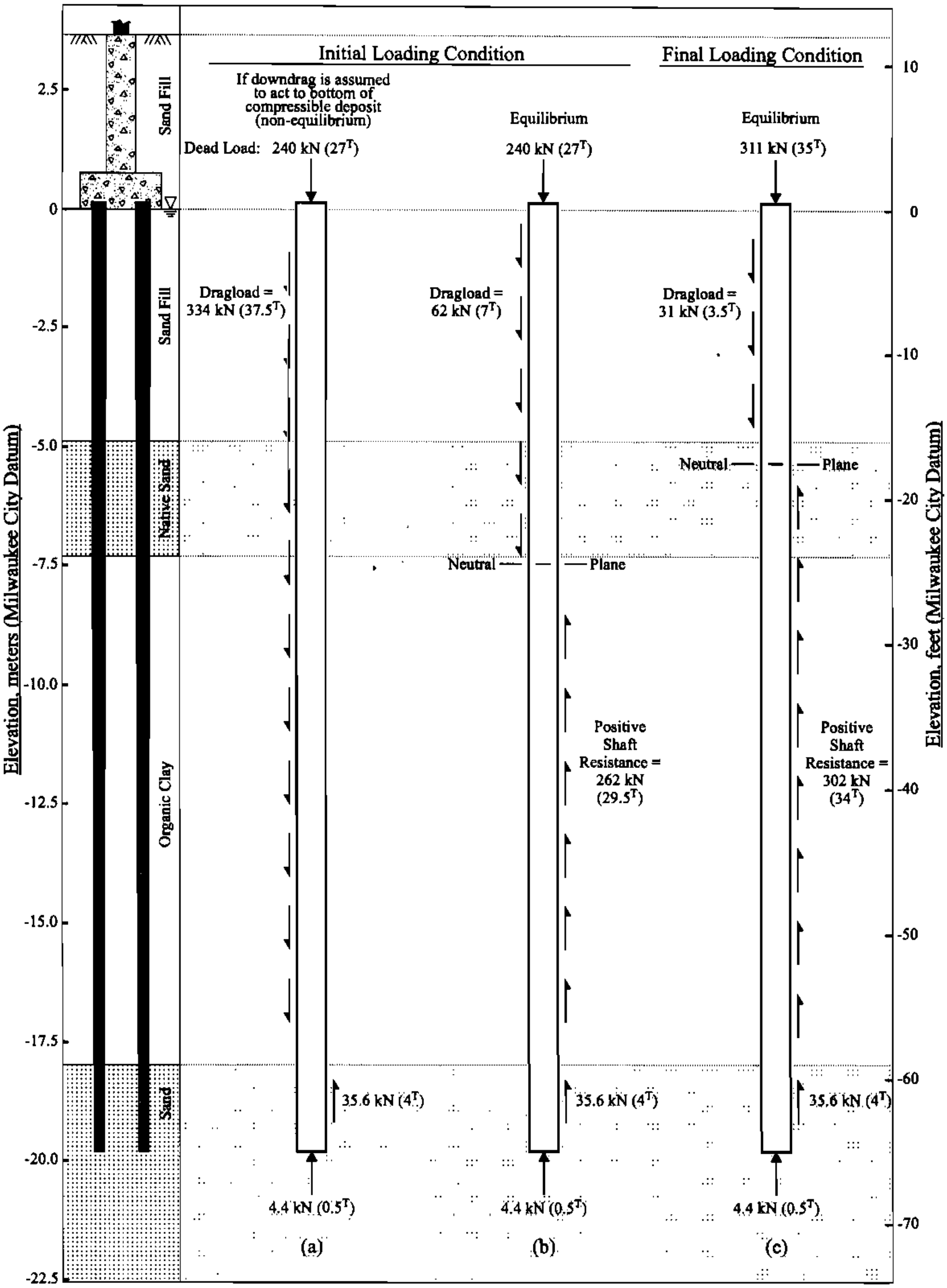

Figure 4. External Pile Forces

positive shaft resistance in the competent soil below the bottom of the organic layer, and 4.4 kN (0.5 tons) of toe resistance, for a total of 40 kN (4.5 tons). Since the downward forces would exceed the upward forces, the pile should (and did) move down. The fallacy of this common model is the assumption that the full dragload continues to act as the pile moves down, which would cause the pile to experience a plunging failure (continued movement at constant load).

In reality, as the pile moved down, skin friction reversed direction in the bottom portion of the pile. In other words, the negative skin friction in this zone became positive shaft resistance (since the soil is settling less adjacent to the bottom portion of the pile than adjacent to the top portion, only the bottom portion of the pile moved downward relative to the soil). The pile continued moving down, and this reversal of forces continued higher up the pile, until force equilibrium was satisfied. It follows from this that downdrag cannot cause the pile to plunge.

At force equilibrium, since the soil was still moving downward relative to the pile along its upper portion, downdrag was still producing dragload, but only above a certain elevation. Below this elevation, positive shaft resistance developed. This concept is illustrated in Figure 4b. The elevation at which these forces changed direction is also the elevation at which the pile and soil settled the same amount, and is called the "neutral plane" [Fellenius]. The neutral plane is located such that the cumulative downward forces in the upper portion of the pile equal the cumulative upward forces in the lower portion of the pile (the pile satisfies force equilibrium). Therefore, the maximum load in the pile occurs at the neutral plane.

The amount that the pile head settles is equal to the soil settlement at the neutral plane, plus the elastic shortening of the pile above the neutral plane (because of its relative magnitude, elastic shortening has been ignored in pile head settlement estimates). A review of Figure 4b indicates that for the initial loading condition (fill and 28.7 kPa (600 psf) floor load), the neutral plane is 7.6 m (25 ft) below the pile head (i.e., at Elevation -7.5 m (–24.5 ft)). A review of Figure 3 indicates that for the initial loading condition, the estimated soil settlement at the neutral plane (and therefore the approximate pile head settlement) is 137 mm (5.4 in). This shows relatively good agreement with typical measured interior column settlements of 142 and 175 mm (5.6 and 6.9 in) for Buildings 3 and 4, respectively (measured 12 years after completion of construction).

As discussed previously, increasing the floor load from 28.7 to 129 kPa (600 to 2,700 psf) increased the total dead load from 240 to 311 kN (27 to 35 tons) per pile. The resulting redistribution of forces acting on the pile is illustrated in Figure 4c. As a review of Figure 4c indicates, the increased pile load led to a 2.1-m (7-foot) rise in the neutral plane, to 5.5 m (18 ft) below the pile head (i.e., at Elevation -5.3 m (–17.5 ft)). A review of Figure 3 indicates that for the final loading condition (floor load increased to 129 kPa (2,700 psf)), the estimated soil settlement at the neutral plane (and therefore the approximate pile head settlement) is 269 mm (10.6 in). This compares to average estimated interior column settlements of 213 and 406 mm (8.4 and 16.0 in) for Buildings 3 and 4, respectively.

Conclusions

1. For the 2 loading conditions evaluated, pile head settlement predictions based on estimated soil settlements and neutral plane locations show reasonably good agreement with measured column settlements.

2. The piles supporting Terminal Buildings 3 and 4 experienced excessive settlement because they did not extend deep enough. For piles embedded only 1.8 m (6 ft) in loose to medium dense sand below the bottom of the compressible organic deposit, the neutral plane for the initial loading condition was located approximately at the top of the organic deposit. For the final loading condition, the neutral plane was located approximately 2.0 m (6.5 ft) above the top of the compressible organic deposit. Significant soil settlement at both these neutral plane locations guaranteed significant pile head settlement for both loading conditions.

3. To reduce settlements to tolerable magnitudes, the piles would have had to extend deeper (beyond the depth explored by the borings). If the piles had been long enough to position the neutral plane at or near the bottom of the organic deposit, settlements would have been tolerable.

4. To position the neutral plane at the bottom of the organic deposit, the piles would have had to extend deep enough such that the toe resistance plus positive shaft resistance below the bottom of the organic deposit equaled the applied dead loads plus the dragload (resulting from negative shaft friction extending from the bottom of the pile cap to the bottom of the organic deposit).

5. Downdrag (resulting from negative skin friction) cannot make piles experience a plunging failure (continued movement at constant load). However, downdrag can make piles experience excessive settlement.

6. All pile design should consider settlement as well as capacity.

Acknowledgments

The authors thank the engineering staff at the Port of Milwaukee for authorizing publication of this paper, and for providing information regarding details of the project. We also thank Mr. Michael Horne, P.E. of Graef, Anhalt, Schloemer & Associates, Inc. of Milwaukee, Wisconsin for providing building survey data and information regarding building loads, Mr. Robert Lukas, P.E. of Ground Engineering Consultants, Inc. of Northbrook, Illinois for reviewing the manuscript, and Ms. Cindy Paul for preparing the graphics included in this paper.

<u>REFERENCES</u>

Fellenius, Bengt H., "Recent Advances in the Design of Piles for Axial Loads, Dragloads, Downdrag, and Settlement," ASCE and Port of New York and New Jersey Seminar, April, 1998.

NAVFAC DM-7.1, "Soil Mechanics," May, 1982.

Wagner, Alan B., "Comparison of Static Pile Analyses and Load Test Results," Proceedings, 16th Annual Member's Conference, Deep Foundations Institute, Chicago, Illinois, October 7-9, 1991, pp. 21-35.