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Failure of an Excavation Support System

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ABSTRACT

The design, construction, and collapse of an excavation support system constructed through layered soils are presented in this paper. The braced soldier-pile and lagging shoring was installed through soft clay, with the base of the excavation in hard glacial deposits. Complicating factors included the use of soil berms for temporary support, construction sequencing, weather conditions, and the location of the failed section near a re-entrant corner of the shoring system. Rapid responses of all contract parties and careful evaluation of the failure causes limited subsequent safety and damage concerns, and no claims were made. Post-failure examination of the preceding events provides several insights into potential better specification practices. Theoretical comparison of soil strength and structural engineering principles demonstrate the true failure mechanisms in spite of several implied causes suggested at the outset of the investigation.

INTRODUCTION

The design, construction, and subsequent collapse of an excavation support system are presented in this paper. During excavation for a subway station, a large section of shoring collapsed. The braced soldier-pile and lagging shoring was installed through soft, varved clay, with the base of the excavation in hard and dense glacial deposits. Complicating factors included the use of soil berms for temporary support, construction sequencing, weather conditions, and the location of the failed section near a re-entrant corner of the shoring system. Due to the particular location and time of the failure, no personnel were injured and damage to other facilities was limited, though conditions were such that immediate actions were required in some other shored areas. Post-failure examination of the events leading to the failure provides several insights into better specification practices. Theoretical comparison of soil strength and structural engineering principles demonstrates the true failure mechanisms in spite of several implied causes suggested at the outset of the failure investigation.

PROJECT DESCRIPTION

Construction of a new subway station required an 11 m deep, shored excavation through a thin layer of fill, varved silt and clay of variable strength (see Figure 1), and a hard cohesive glacial till underlying the soft deposits. Undrained shear strength of the hard glacial till deposit was about 300 kPa or greater.

The contract for the subway station was developed as a design-bid-build contract. A performance specification governed the design of the shoring system; minimum design and construction requirements were to be met, but the detailed shoring design was to be completed by the contractor. A subcontractor completed shoring construction and a separate firm retained by the shoring subcontractor completed the shoring design. Excavation was carried out by a separate organization subcontracted to the general contractor.

GENERAL SHORING DESIGN

The soldier pile and lagging shoring system consisted of steel wide-flange beams placed in pre-drilled holes with the annular space around the beams filled with a sand-cement mix (unconfined compression strength $\approx$ 0.4 MPa). The collapse occurred at a corner of the shoring where the station excavation changed in width. The planned shoring design is illustrated in Figures 2 and 3. In this area, the horizontal restraint for the narrow section was to be provided by pipe struts running north to south where resistance could be provided by the opposing side of the excavation. In the wider station area, support was to be provided by both corner braces and tiebacks. Existing utilities diagonally crossing the excavation were relocated so they paralleled the shoring line.

SEQUENCE OF SHORING CONSTRUCTION

After relocation of the utilities and installation of the piles, the

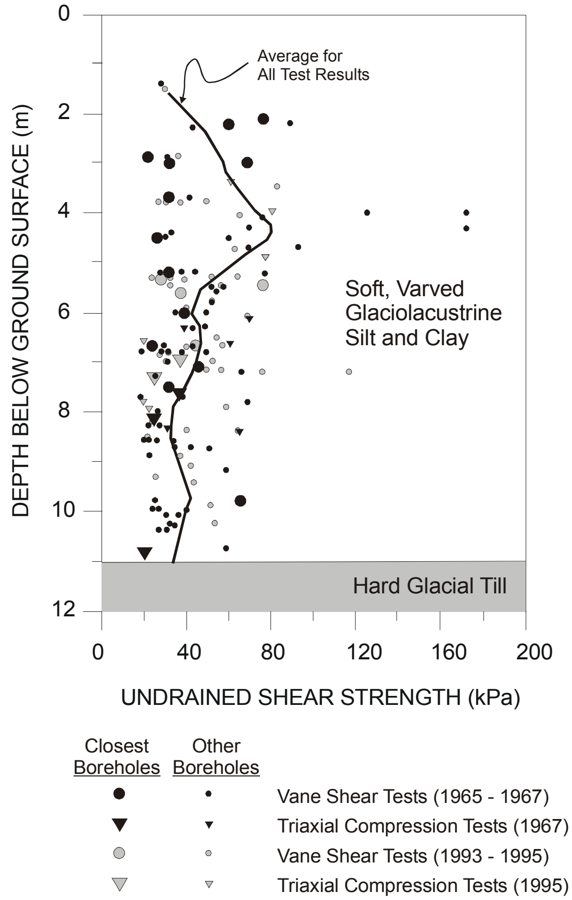

Figure 1. Subsurface conditions in area of shoring collapse.

excavation was taken down to the level of the first tieback row, about 2 m below the top of the piles, and the tie-backs were installed. After about a month and a half, excavation was resumed and taken to the level of the second row of tiebacks and the tiebacks were installed, but not connected to the shoring or stressed. A labor strike took place, delaying further construction for about three weeks. Following the strike, the work resumed as follows:

Day 1. Excavation resumed at Structure Unit 79 near the southwest corner, and only the excavation subcontractor was working.

Day 2. Excavation and lagging continued between piles S81 and S92 for Structure Unit 81. A ramp for excavation equipment was prepared on the north side of the excavation, directly opposite piles S87 to S90. Sewer backfill and native soil collapsed through lagging between piles S80 and S81 at 5.5 m below the ground surface north of the manhole.

Day 3. The center and north side of Structure Unit 79 was excavated. Lagging was installed between piles S73 to S109. Wales were installed between piles S82 and S87.

Day 4. The entrance gate for earthmoving equipment was relocated to the north side ramp location. Excavation continued and lagging was installed between S97 to S107.

Day 5. A wale was installed between piles S80 and S82 and excavation and lagging continued between piles S97 and S101.

Day 9. Following the weekend and holiday, excavation was resumed. The lower tie-backs were stressed from pile S76 to S82, inclusive. Earth berms were left in place to support the retaining structure prior to placement of struts, as illustrated in Figure 4. This practice is typical of local construction, and is a practice used elsewhere (e.g. Clough and Denby, 1977).

Day 10. Excavation continued on the north side of Unit 79 and 88 and lagged between piles S71 to S83 and N73 to N81 (not shown in Figure 2). Figure 4 illustrates the conditions on Day 10 prior to failure.

Day 11. Excavation continued on north side of Structure Unit 79, but no lagging or welding occurred due to rain.

Day 12. No work due to rain.

Day 15. After heavy rain over the weekend, excavation continued in center of Structure Unit 79. Struts were installed across the excavation at piles S91 to S96. Lagging and excavation continued between piles S71 and S81 and was finished to subgrade, leaving one lift of lagging to be completed from piles S83 to S87.

Day 16. The shoring collapsed in the early morning hours prior to crews arriving on site. Figure 4 illustrates the conditions immediately prior to the collapse and Figure 5 illustrates conditions on the morning the collapse was discovered.

WHAT WENT WRONG?

Immediately following the collapse various parties suggested several possible causes. The suggested causes included:

- heavy rainfall contributing to additional loads (filling of the sewer trench backfill);
- deep-seated failure of the ground beneath the shoring occurred because of higher pore water pressures and low strength; and
- a lack of structural or earth support.

A detailed evaluation of all of these potential causes was undertaken to ascertain the possible failure mechanisms. It was considered necessary to evaluate the failure to determine what actions, if any, had been taken or ignored that precipitated the

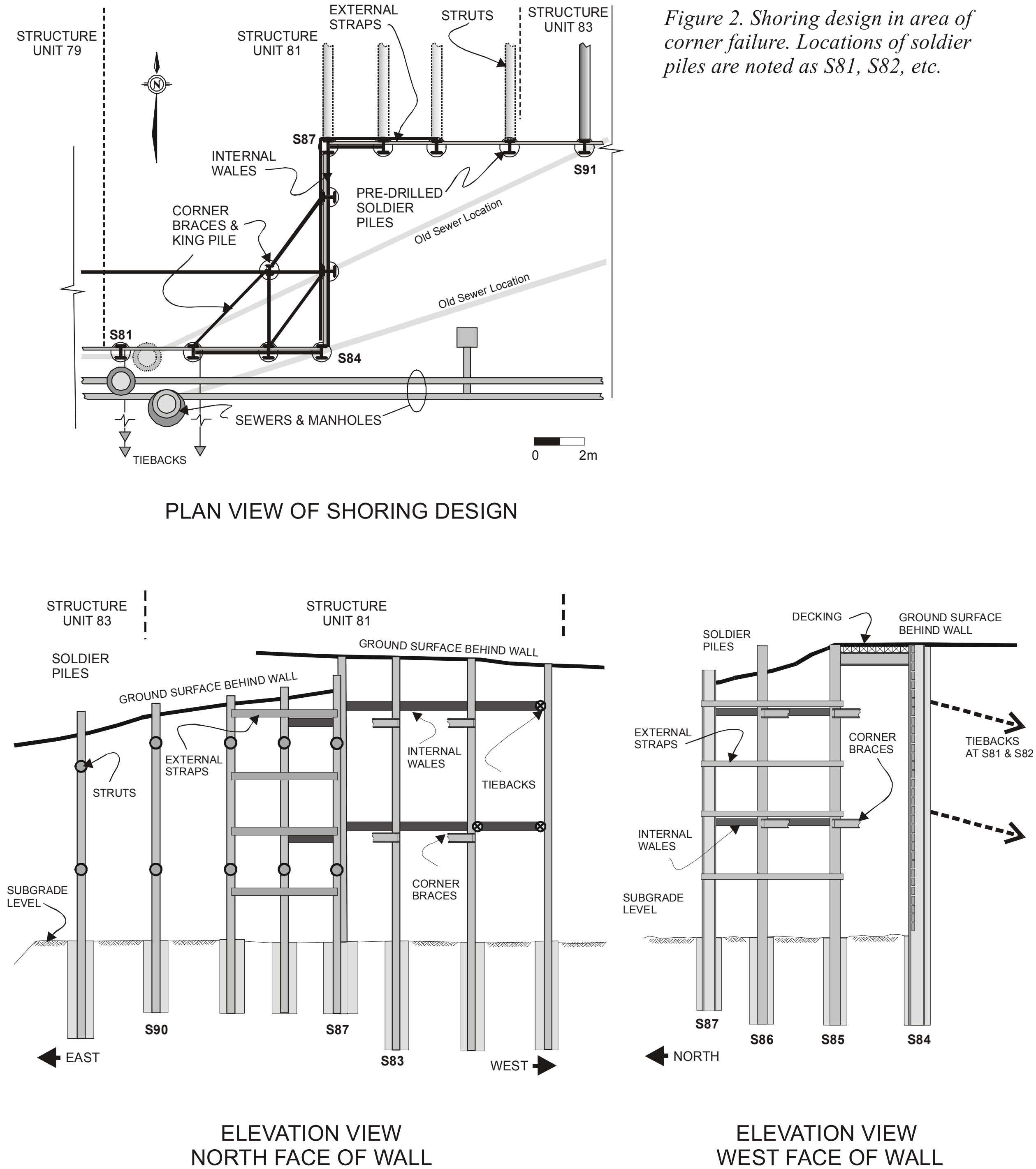

Figure 2. Shoring design in area of corner failure. Locations of soldier piles are noted as S81, S82, etc.

failure to avoid similar situations occurring elsewhere on the project. Where possible, the most unambiguous analysis methods and averaged soil properties were used to provide as much of an unbiased conclusion as possible.

To understand what went wrong, the simplest of these causes was examined first, namely, the possible lack of structural or earth support.

<u>Loads and Resistance</u>

The failure of the shoring system involved complex three-dimensional loading and three-dimensional variations in the load resistance characteristics of the support system components. Therefore, the loads and resistances were resolved into orthogonal components in the north-south (NS) and east-west (EW) as schematically illustrated on Figure 3.

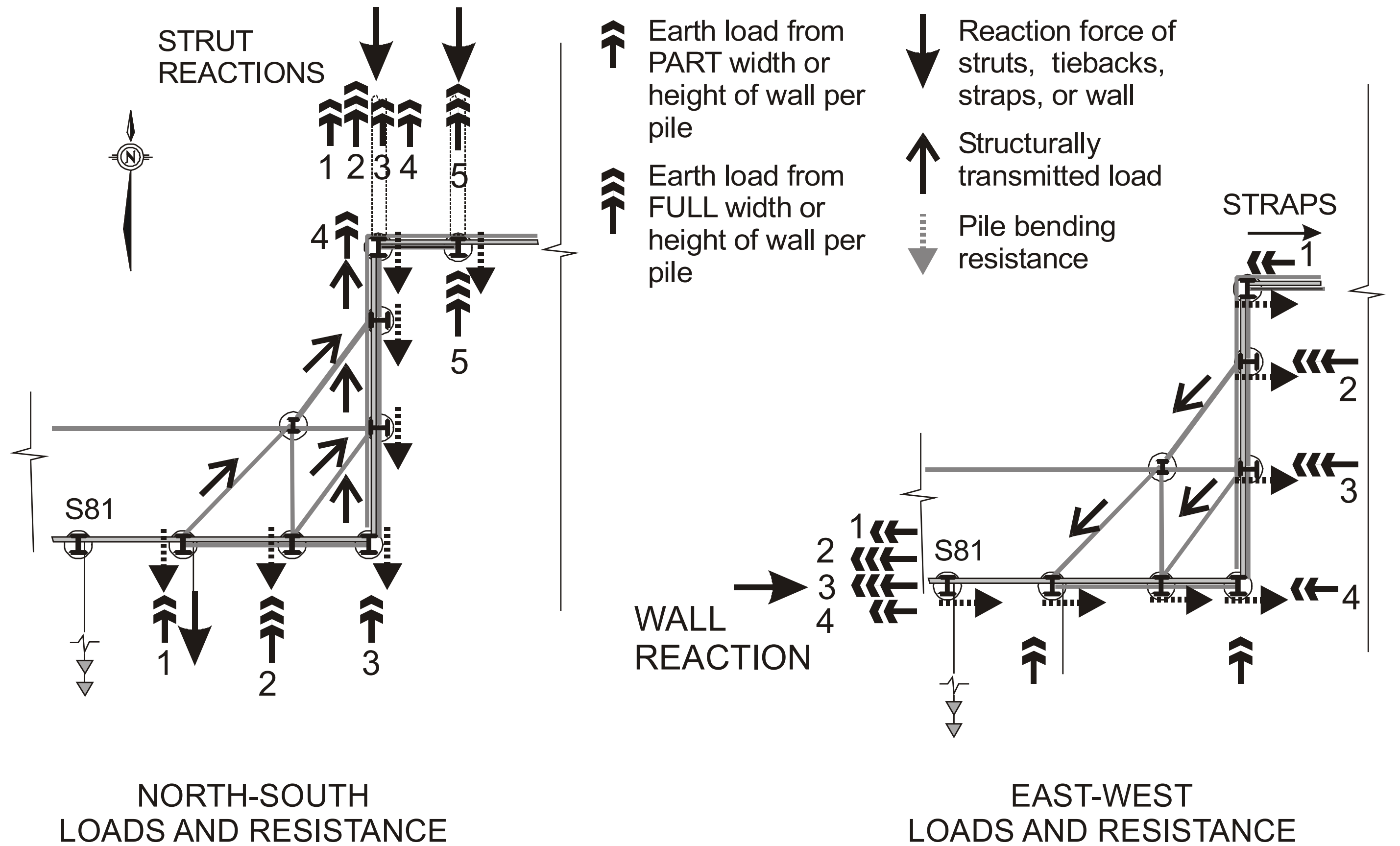

Figure 3. Earth loads and support mechanisms for shoring corner had all structural members been in place.

The NS load component in this case has to be resisted ultimately by the struts placed against pile S87 and the bending resistance of the piles S82 through S87. Prior to the failure, the contractor's personnel and advisors considered that the northward loads would be resisted by earth berms left in place in front of piles S87 to S90.

The conditions and geometry of the failed earth mass behind the shoring were generally consistent with an active failure of the earth mass behind the wall with subsequent progressive failure of the unsupported soils. The maximum distance from the wall to the failure scarp was approximately 1 to 1.5 times the maximum vertical height between the glacial till layer and the ground surface elevation. The scarp formed a circular patter with a radius originating near the location of the corner pile (S87). All of the piles destroyed by the failure also appeared to be bent in the direction of failure near the interface of the glacial till and glaciolacustrine soils.

Undrained soil parameters were used in the analysis for the cohesive soils because of the relatively short duration of excavation and subsequent failure. A plot of the undrained shear strength measurements through the clay deposit is illustrated in Figure 1. For granular soils, such as those placed in the replacement sewer trench and as backfill in the old sewer trench, typical effective-stress frictional properties were used in the calculations.

Because the piles were loaded in cantilever bending, i.e. there were no struts in place, and deformations were not restricted, active and passive earth pressures were assumed to act on the shoring system. Active earth pressures were determined using the commonly-used equations of Bell (1915) for a cohesive soil (no frictional component) and a zone exhibiting zero active earth pressure in the upper part of the wall (though the tension component of this load was ignored). This approach produced the minimum active load on the wall. The presence of the sewer trench and backfill (partially saturated or fully saturated) was also evaluated using conventional earth pressure methods where the trench was considered equivalent to a partially water-filled or fully water-filled tension crack (e.g. Bowles 1996). The passive earth loads and resistances were determined using trial wedge methods (e.g NAVFAC 1986, Kezdi 1979), particularly because of the downward back-slope geometry of the berm. The soil load or resistance on any one pile was taken as the pressure or unit load multiplied by the total horizontal distance halfway to the piles on either side in the appropriate analysis direction. These approaches were adopted for their simplicity of calculating acting and resisting forces and their resultants using unambiguous and well-know formulae. At the time this work

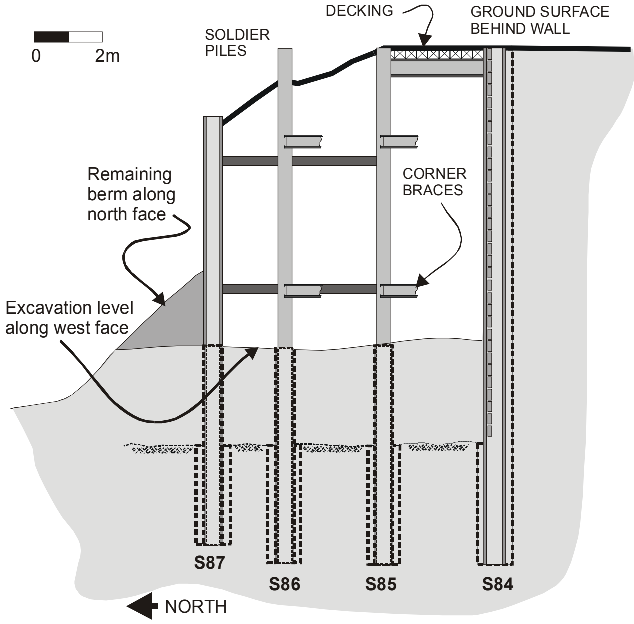

Figure 4. Conditions prior to failure.

was completed, there was insufficient time for extensive numerical modeling of this problem.

From the available evidence, it was clear that the piles were loaded in a cantilever mode. Cantilever "beams" composed of wide-flange steel sections are inherently unstable unless adequate support is provided to the flange loaded in compression (in this case, the exposed face of the piles). Wide-flange steel sections are unstable under cantilever loading conditions because their resistance to bending in one direction (the strong or X axis) is far greater than in the perpendicular direction (the weak or Y axis direction). Lateral bracing of the compression flange of shoring piles is generally achieved with: 1) nominal passive resistance provided by the lagging, although the lagging is not directly connected to the piles; and 2) the strength of the soil (and filler material for pre-drilled piles) that surrounds the pile. The degree of lateral bracing of the piles involved in the shoring collapse was questionable because the steel support in the plane of the shoring face varied in location and connection detail. It was considered that the lagging would not give any lateral support to the piles once failure began since as the piles deflected, the lagging would likely cease to be in contact with the steel and would be free to fall away from the shoring. Therefore, the ultimate bending resistance of the piles was calculated according to the methods outlined by Johnston (1976) and CISC (1993) and was calculated for unbraced lengths (L) ranging between 0 and 10 m. It was considered that beam tables and code-based formulae were insufficient for evaluation of ultimate structural capacity in unbraced bending in both strong and weak axes as lateral-torsional buckling could readily dominate the failure mode. The formula for the ultimate or critical stress induced by lateral-torsional buckling for a cantilever beam is (Johnston, 1970):

$$\sigma_c = \frac{C_1\pi(EI_yGJ)}{S_x(KL)}\left[1 + \frac{\pi^2 EC_w}{GJ(KL)^2}\right]^{1/2} \qquad (1)$$

where the elastic and shear moduli (E and G), torsion/warping constants (J and C_w), internal moment of inertia and section

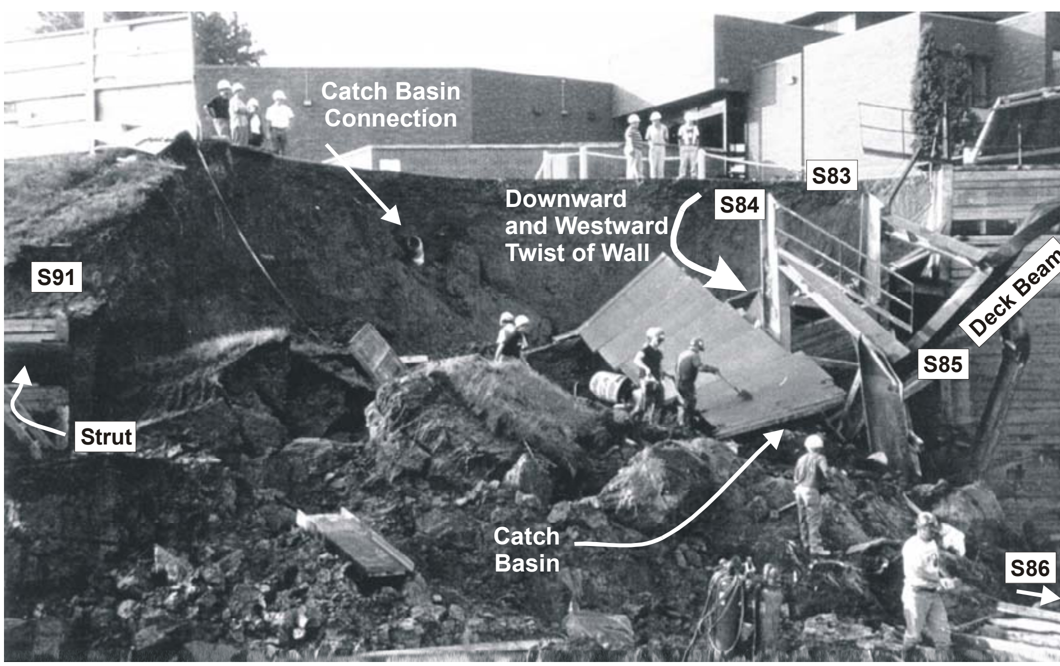

Figure 5. Photograph of collapse looking south.

modulus (I and S) are found in steel design manuals (e.g. CISC 1993). The constant K, for a fully laterally unsupported cantilever beam with one end fixed in both axis directions, is set equal to one. The constant C_1 ranges from 1.3 for a concentrated load to 2.05 for a uniform load. For this project, it was considered that the loading would be closer to uniform than a concentrated point load, and this constant was therefore set equal to 2.05 to result in the maximum calculated resistance.

Analysis Cases

It was clear that all of the structural support designed for the shoring system was not in place at the time of failure. Key components of the support that were missing included the steel straps along the north face (piles S87 to S91) and the struts against piles S87 through S90. As noted, it was believed by the contractor and shoring designer that the earth berm would provide sufficient temporary support. For the conditions immediately prior to failure, three separate analysis cases (see Figure 6) were examined to determine which of the possible load and resistance modes may have triggered the failure.

Case A: Construction up until the time of failure had not included installation of the steel straps along the north face, between piles S87 and S91, as indicated in Figure 2 and 4. In this case, there was no support to pile S87 in the east-west direction except for the pile's own bending resistance.

Case B: The absence of struts placed against pile S87 at the time of the failure meant that only the cumulative bending resistance of the piles was available to resist the northward component of the loading on piles S83 to S87.

Case C: North-south loading was also examined for pile S88 as representative of piles S88 through S90 that also failed. Load resistance for these piles was to be provided by the two levels of struts that spanned the excavation to the north side. Because the struts were not in place for these piles, the load resistance was provided by the earth berm and bending resistance of the piles.

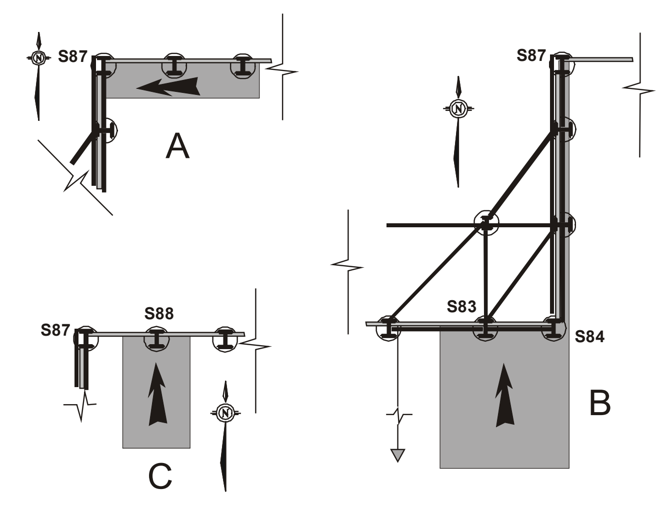

Figure 6. Analysis cases.

Analysis Results

Figure 6 illustrates the loading conditions on the piles for Cases A, B, and C. The point on the "unbraced length of pile" axis, below the intersection of the lines plotted for structural bending resistance and the lines plotted for soil loading indicates the point at which pile collapse could be expected to occur (critical point). In assessing the results of the analyses, the lateral bracing conditions were considered to be between the two extremes of unbraced length where partial bracing was present, e.g. pile S87 where the internal wales are present on the south side but no support is provided on the north (see Figure 7). As illustrated in this figure the calculated critical point for Load Case A occurs at an unbraced length less than the possible field conditions. The difference between the calculated critical point and the smallest of the possible unbraced lengths for Case A is greater than that of either Cases B or C. The results clearly indicated that collapse of the piles due to static loading would be likely for both load Case A and B, with Case A being the most critical. There is greater uncertainty with respect to soil strength and earth pressure loading than for the structural conditions at failure, particularly considering the effects of time. It is therefore considered that the actual active earth pressure acting on the wall at the time of failure was slightly less than or the passive earth resistance slightly more than assumed in the structural analysis (see Figure 7).

Based on the observed conditions immediately after failure and subsequent analyses, it was concluded that the principal engineering causes of the failure were, listed in order of importance:

1. plate steel straps were not in place along the north wall face between piles S87 to S89;
2. struts were not in place to support piles S87 to S90 for the convenience of the north-side excavation ramp;
3. excavation and lagging was rapidly completed to within 2 m of the subgrade elevation all along the west wall; and
4. the center of the excavation was taken to approximately 1 m above the final subgrade elevation leaving only earth berms to support the piles on the south side of the excavation.

It was considered most likely that, because of the rapid excavation along the west wall, inadequate support was provided to pile S87 in its weak-axis direction and this pile became overloaded. Without the presence of the steel straps along the northern face, the bending resistance of the single pile in its weak direction was insufficient. Failure of this pile, triggered further instability of the piles along the west wall, leading to collapse of the southwestern section where the wall was not restrained by tiebacks. Simultaneously, the collapse of pile S87 precipitated deformation and removal of a portion of

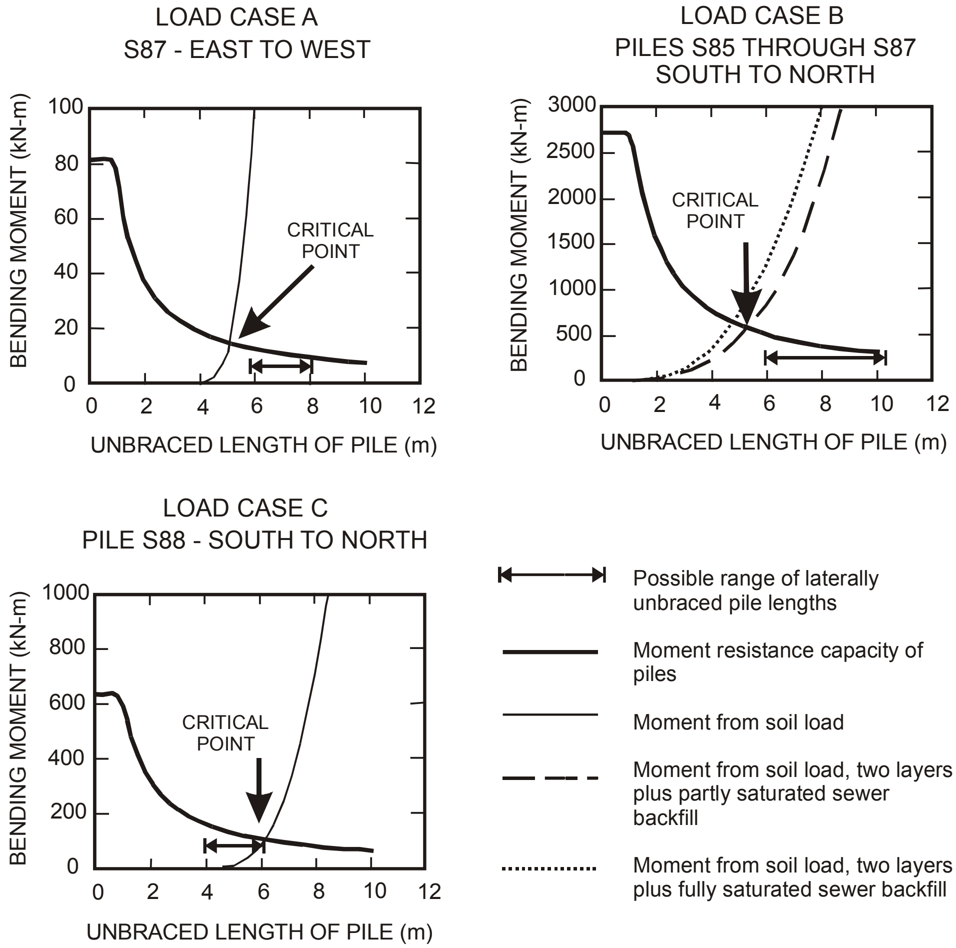

Figure 6. Comparison of cumulative ultimate lateral-torsional pile bending moment capacity to moments induced from soil loads.

the remaining berm on the north side between piles S87 and S88. Successive west to east loss of ground forming the berm and any nominal lateral bracing provided by the lagging, as each subsequent pile eastward collapsed, is likely to have resulted in the progressive failure of the shoring system. Failure stopped where pile S91 was restrained by a strut and S82 was restrained by a tieback.

On Figure 7, the influence of rainfall saturating the sewer trench backfill is also illustrated. It can be seen that the potential filling the trench with water likely had little if any effect on the overall stability of the shoring and ground system.

Analyses were also conducted to examine the potential for deep, rotational stability to satisfy the concern that failure near or below the level of the pile toes precipitated the shoring collapse. These analyses indicated that the overall ground mass would have exhibited a factor of safety of much greater than 3 had the wall remained intact.

CONCLUSIONS

Theoretical formulae for the calculation of ultimate lateral-torsional buckling, active earth pressures, and passive resistance agreed well with the conditions preceding the failure. Although there was some unknown influence of the actual distribution of stresses, the actual unbraced pile lengths, the coefficients chosen for structural calculations, and the simplified resolution of loads into two directions, the analytical results also indicated a reasonable mechanism for overall failure of the support system.

The failure of an engineered system is seldom due to one critical mistake. During the days immediately preceding the collapse, several factors combined to lead to the failure including:

- the strike led to impatience and accelerated construction;
- the weather inhibited work of the shoring subcontractor's crews and created more impatience;
- the excavation subcontractor was not under the direct

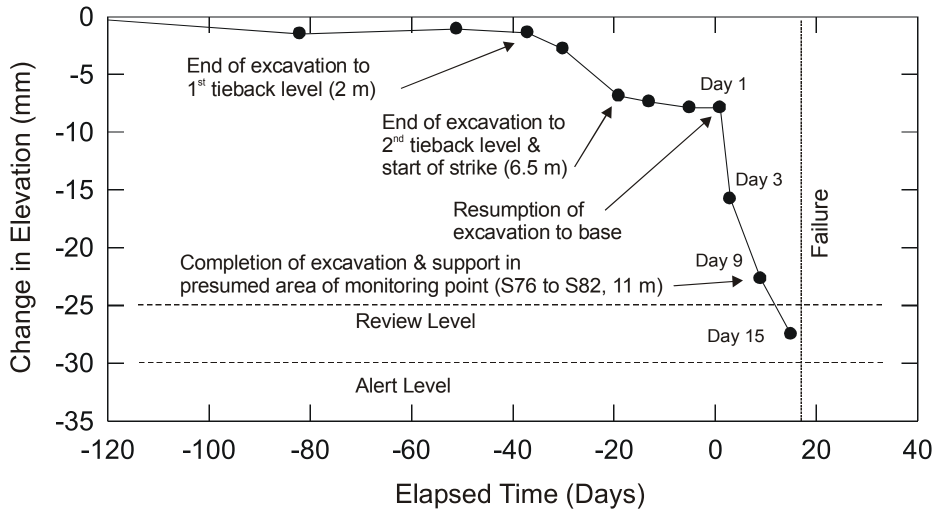

Figure 8. Settlement monitoring data for mislabeled survey point located 3 m east of S84, previously presumed to have been at the position of another point located between S81 and S80 (labels were mistakenly switched in the field).

control of the shoring subcontractor;

- aggressive excavation and disagreements related to the equipment access ramp confused construction in the immediate area;
- other activities demanded the attention of the shoring foreman and on-site geotechnical representative; and
- one of the monitoring points in the immediate area of the collapse had been mislabeled and was thought to be in an area already fully supported (see Figure 8) and, according to design expectations, the measurements had not reached a previously determined critical level until one reading was taken late in the day immediately preceding the failure.

These issues contributed to the lack of attention to detail regarding installation of the steel straps and maintaining earth support in critical areas. In future contracts, it would be beneficial to require that the excavation crews be under the direct supervision of the shoring foreman.

Considering the activities preceding the failure, it was exceptionally fortuitous that no one was seriously injured as the result of the failure. It must also be recognized that, although many factors contributed to the failure, all of the contractors and subcontractors worked rapidly and without self-interest in order to repair the failure and resume work. Exceptional cooperation among all on-site personnel resulted in a project that, in spite of the shoring failure, became a success for all involved. Rapid responses of all contract parties and careful evaluation of the failure causes limited subsequent safety concerns and damages, and no claims were made.

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