Braced Excavation at the NIPSCO Bailly Station Power Plant

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SYNOPSIS: In July 1991, the intake and discharge pipelines of a major power plant collapsed. A 60-ft. deep excavation adjacent to several structures sensitive to ground movements was required for remediation. Based on conventional analyses, the estimated factor of safety against base heave was close to 1.0 for the required excavation, and there was grave concern for damage to appurtenant structures. A viable reconstruction scheme was developed through the integration of finite element analyses and construction monitoring.

INTRODUCTION

The Bailly Generating Station is a coal fired power plant that started producing electricity in the early 1960's. The plant is owned and operated by the Northern Indiana Public Service Company (NIPSCO) and is located on the southern shore of Lake Michigan north of Chesterton, Indiana. On July 2, 1991, the 14-ft. diameter intake and discharge pipelines suddenly collapsed, causing a complete shutdown of power production. In addition, critical pollution control structures were located within and immediately adjacent to the resulting sinkhole, which measured approximately 85 ft. wide and 20 ft. deep. Due to the financial losses accruing daily from the inoperable power plant, a fast-track reconstruction scheme that would quickly replace the buried pipelines without damaging sensitive appurtenant structures was implemented. Reconstruction required an excavation approximately 50 feet wide, 625 feet long and 60 feet deep. This paper describes how construction monitoring was used in conjunction with finite element analyses (FEA) to guide the design and construction sequence of the excavation support system comprised of driven steel sheet piles braced by both cross-lot struts and tie-back anchors.

The paper mirrors the sequence of events that evolved. The project is described with emphasis on the often conflicting needs of minimizing both construction time and ground movements. The general subsurface conditions were assessed and a preliminary bracing system designed using conventional limit equilibrium analyses. Design alternatives were investigated using FEA. As construction commenced, additional borings and soil tests were made. Design of the sheeting and bracing system, as well as construction procedures, were successfully modified based on continual upgrading of the input to the FEA. Construction monitoring assisted in validating the recommendations made based on the FEA and in assessing the safety of the bracing system and the potential for damage of adjacent structures due to construction-induced subsidence. The limitations imposed on the length of the cut at the bottom of the excavation, the rapidity with which struts and anchors were installed and prestressed, and the high standards of quality control that were maintained contributed to the "better-than-expected" performance of the retaining system.

PROJECT DESCRIPTION

Using normal work schedules and conventional approaches, reconstruction of the water cooling system would have required 18 to 24 months. Even a fast-track construction time of 5 months could potentially create large financial losses. The over-riding priority during this project, therefore, was to minimize the time necessary to resume power production.

The reconstruction project (see Fig. 1) consisted of replacing two parallel corrugated steel pipes, 14 feet in diameter, 625 feet long, with inverts as much as 60 feet below ground surface. Over 60,000 cubic yards of soil would have to be excavated and replaced adjacent to several structures sensitive to subsidence. Except for a tall stack, these structures were supported by shallow foundations. Damage to the adjacent structures would cause intolerable delays in implementing a full-scale pilot study of an innovative process for pollution control.

The sudden collapse of the 14-ft. diameter pipelines resulted in an extensive surface depression. As the volume of the depression exceeded the volume of collapsed pipe, subsidence was due in part to sand being washed into Lake Michigan. Facilities overlying the subsidence zone were heavily damaged, as shown in Figure 2. A system of trusses used to support an extensive duct network collapsed. Some of the footings supporting the trusses dropped as much as 20 feet with foundation anchor bolts and bottom plates failing in tension. The differential settlement across a large masonry structure 40-ft wide was over 15 feet. Remarkably, no cracks were observed in the masonry walls of this building. A 100 ft. by 110 ft. mat foundation supporting an absorber building extended unsupported for a distance of approxi-

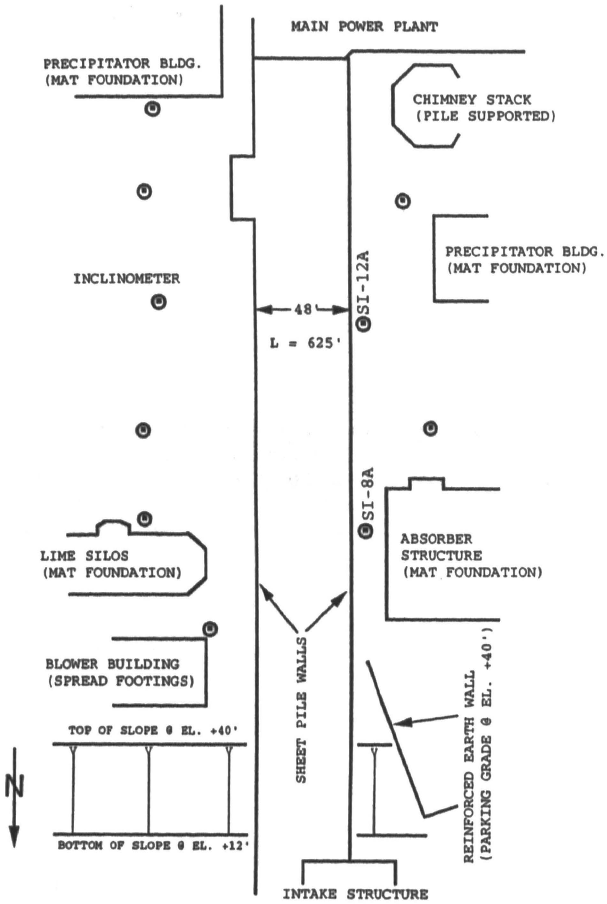


Fig. 1 Bailly Generating Station Site Plan

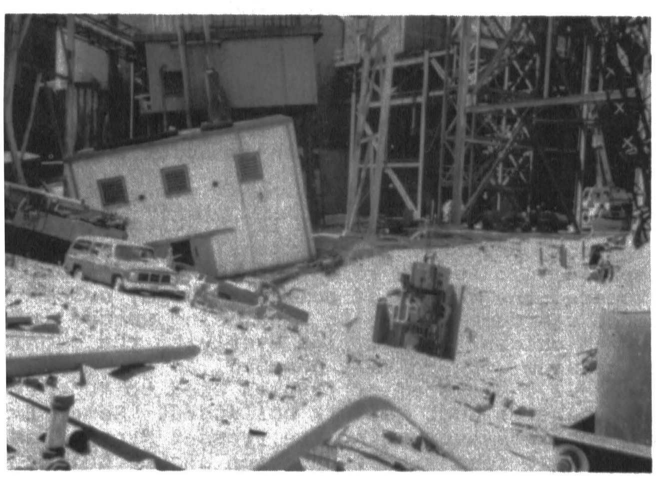


Fig. 2 Surface Deformation Resulting from Pipeline Collapse

mately 20 feet over the subsidence bowl. Saving this structure was of primary importance.

SUBSURFACE CONDITIONS

The Bailly Generating Station site lies in the Calumet Lacustrine Plain, which is an area of generally low relief that occupies the former lake bed of glacial Lake Chicago. Sediments of the Calumet Lacustrine Plain consist of a variety of materials, including lacustrine clay and silt, deposits of muck and peat, expanses of beach sand with accompanying sand dunes, and clay rich till units of varying thicknesses. A generalized subsurface profile oriented along the longitudinal axis of the excavation is shown in Figure 3(a). A typical transverse cross section is shown in Figure 3(b). The original ground surface adjacent to the power plant is typically at elevation +40 feet with respect to mean lake level. The natural ground water level near the power plant is at approximately elevation +10 feet.

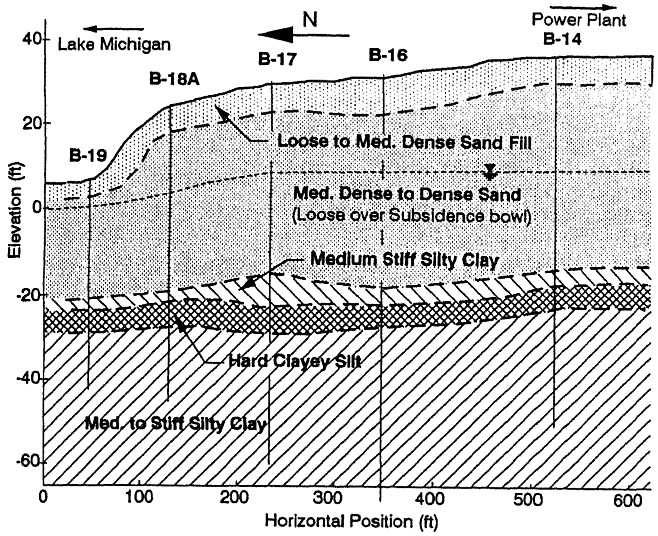
The near surface soil conditions at the site consist of loose sand fill of variable thickness overlying medium dense to dense sand extending to approximately elevation -10 feet. Typically a layer of medium stiff silty clay is present beneath the sand layer extending to approximately elevation -20 feet. A hard clayey silt stratum of thickness ranging between 4 and 10 feet underlies the clay. Below the hard silt layer is a medium stiff to stiff silty clay which extends down to hard glacial till at an approximate elevation of -70 feet. The hard glacial till overlies dolomitic limestone which is present at approximately elevation - 140 feet.

DESIGN CRITERIA

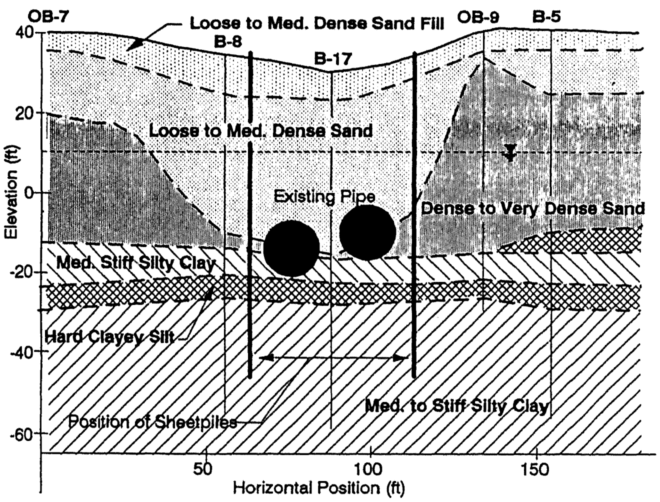
During the three days following the collapse, design concepts were evaluated to determine a suitable earth support system for replacement of the intake and discharge pipelines. Utilizing existing soil boring logs, initial design concerns for the earth retention system were outlined:

- The potential for base heave instability (Terzaghi 1943) in the silty clay located at the base of the excavation was high.
- The bracing near the bottom of the excavation would require a vertical spacing of approximately 20 feet to accommodate the 14 foot diameter pipe and the 4 foot difference in invert elevations.
- A stiff retention system would be required to limit ground settlement. Critical adjacent structures with bearing pressures ranging from 1000 psf to 4000 psf were located between 20 and 100 feet of the excavation. Allowable differential settlements were initially estimated to be between 1/8 to 1/4 inch.

Due to the limited accuracy in estimating de-



(a) Longitudinal Cross-section



(b) Transverse Cross-section

Fig. 3 Generalized Subsurface Profile and Cross Section

formations using empirical methods, the FEA was utilized to provide additional insight. The proposed system was analyzed to assess the approximate magnitude and shape of the wall movements and surface settlements, the structural loads in the wall and bracing system, and the stability of the base soils. Two-dimensional FE results using the preliminary soil data indicated little possibility of achieving the stringent differential settlement criteria of less than 1/4 inch. Pressure grouted pin piles were installed below the mat foundations of two critical structures located immediately adjacent to the excavation. The pin piles were installed with a jacking system to allow compensation of subsequent settlements due to strains in the underlying clay. With these provisions, the surface settlement criterion was relaxed to 1 inch.

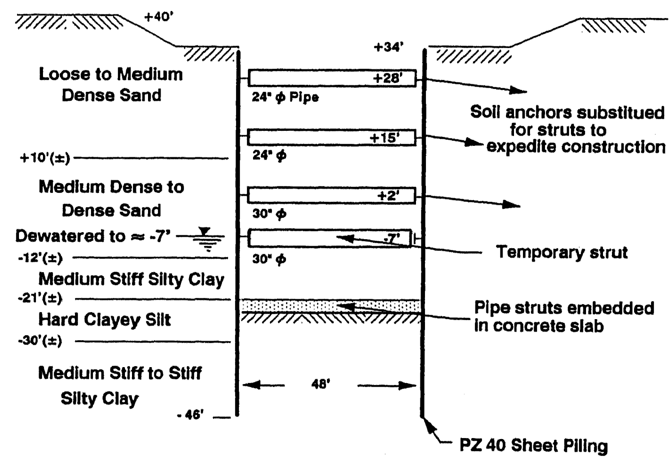


Fig. 4 Braced Excavation Design

BRACING DESIGN

The preliminary design consisted of an internally braced steel sheet pile system. Steel sheet piles were selected because of availability and because of its adaptability to the schemes of bracing and dewatering under consideration (e.g. well points through the sheeting). PZ40 sheet piling, the heaviest section modulus sheet piling available (60.7 in^3/ft.), was selected in 80-ft. lengths to provide embedment below the excavation base equal to one half the excavation width. The manufacturer established a special rolling for the material and the sheet piling started arriving onsite three weeks after the pipelines collapsed.

The preliminary design of the proposed retention system was developed using apparent pressure diagrams that included soil, surcharge and water pressures. The top of the sheet piles were positioned at elevation +34 feet. The original proposed internal bracing system consisted of five rows of struts with specific vertical locations determined as follows (see Fig. 4):

- A row of struts would be located just above the crest of the discharge pipe at Elev. +2 ft. (Strut S3). Two additional rows of struts would be located above the pipelines at Elev. + 28 and + 15 ft. (S1 and S2, respectively).

- Due to the large vertical span required to install the pipe sections (20 ft.) and the lower strength clay stratum below the sand, a temporary row of struts would be installed at Elev. -7 ft. (S4).

A final row of bracing below the invert elevation of the intake pipeline was established at Elev. -20 ft. (S5). The initial design proposed placing segmental concrete slabs at this level.

Analysis of the proposed sheet pile and bracing system indicated that the sheet piles would be overstressed in bending upon removal of the temporary row of struts at Elev. -7 feet. Consequently, the natural ground water level was lowered by pumping from Elev. +10 ft. to -7 ft. to reduce the sheet pile bending moment.

The initial design called for 24 and 30 inch diam. pipe (with a half inch wall thickness) as cross-struts spaced on 20 feet centers. The geometric constraints of this bracing system would require the use of a clam shell bucket for all excavation and was considered to be too time consuming. On the other hand, the use of soil anchors to replace struts S1, S2 and S3 would greatly expedite the construction process as excavation could proceed unimpeded by the cross-struts.

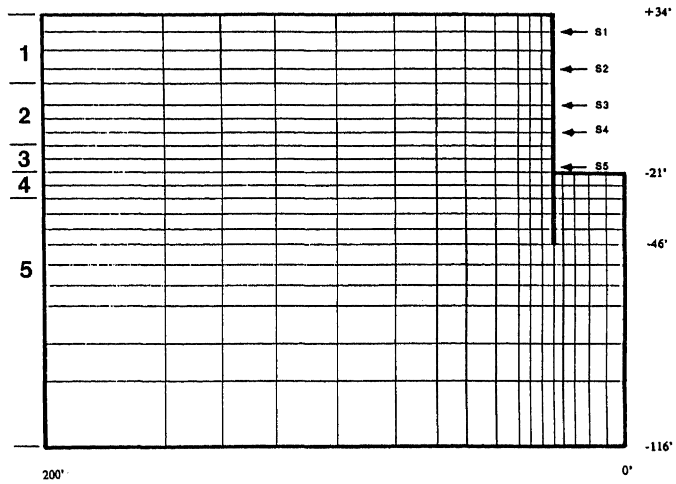


Fig. 5 2-D Finite Element Model

TABLE I. Hyperbolic Soil Parameters Used in Baseline FEA

SOIL LAYER (1)	C (psf) (2)	ϕ (deg.) (3)	K (4)	n (5)	R_f (6)	K_b (7)	m (8)
1 - FILL	0	34	600	0.5	0.8	350	0.2
2 - SAND	0	42	800	0.5	0.8	350	0.2
3 - MED. STIFF CLAY	900	0	500	0.011	0.88	8,500	0
4 - HARD CLAYEY SILT	4000	0	1000	0.011	0.88	17,000	0
5 - STIFF CLAY	1800	0	680	0.011	0.88	11,560	0

FINITE ELEMENT MODELS

Preliminary FEA were performed to check the estimates of strut and anchor loads while the site investigation was being conducted. The main purpose of the FEA, however, was to estimate the expected wall movements and surface settlements, and these analyses were upgraded as additional soil information became available. Two FE programs were utilized to provide insight into the likely performance of the bracing systems. Two-dimensional (2-D), plane strain models were analyzed with the program SOILSTRUCT (Filz et al. 1990) which employs the Duncan et al. (1980) hyperbolic soil model, and the reduction of surface settlements due to three-dimensional (3-D) effects from shortening the allowable open excavation length was evaluated using the program CRISP (Britto and Gunn 1987).

Given the time constraints of this project and the limited soil data available during design, SOILSTRUCT offered a reasonable compromise between the sophistication and efficiency required to provide both valid and timely insights. The baseline 2-D FE mesh analyzed in this study is shown in Figure 5. Nearly 400 five-node subparametric, quadrilateral soil elements were used to model the soil, and 15 Euler-Bernoulli beam elements represented the sheet pile wall with its bending resistance. The incremental simulation of the actual construction sequence included soil excavation, application of preloads, and strut or tie-back installation.

The Duncan et al. (1980) hyperbolic soil parameters were developed for each of the five significant soil strata (see Fig. 4), and the model parameters for the baseline case are presented in Table I. These parameters were developed based on the preliminary soil data which included SPT blowcounts and unconfined compression tests on shelby tube soil samples. Due to the uncertainties involved with evaluating soil properties, a sensitivity study was performed. For example, the hard clay layer (layer 4) was assigned undrained shear strengths within the range of 2000 psf and 4000 psf to assess the importance of this layer in minimizing lateral wall movements. Likewise, the undrained shear strength of the base clay (layer 5) was varied within the range of 1000 psf to 3000 psf to evaluate its effect.

By controlling excavation sequences, the surface settlements could be reduced by taking advantage of 3-D effects. Deformation magnitudes of both the excavation base and the ground surface were compared for axisymmetric and plane strain conditions with the program CRISP using a linear elastic soil model. For a 50-foot diameter circular excavation, base heave estimates were approximately the same as the 2-D model but the ground surface settlement was substantially reduced. Accepting the qualitative nature of this approach in assessing 3-D effects, the plane strain analysis for surface subsidence using the program SOILSTRUCT were conservatively reduced by approximately 30 percent based on this conceptual analysis.

PRELIMINARY FINITE ELEMENT RESULTS

Wale Loads

The FEA provided the estimated wale loads presented in Table IIa. Initial wale load estimates based on apparent pressure diagrams and the assumption that the sheet piling was simply supported between the wales are also presented in Table IIa. The estimated wale loads are essentially the same at the first two strut levels. At the critical third strut level, the FE results estimated wale loads approximately 100 percent greater than the apparent pressure diagram when the temporary bracing (S-4) was in-place and 25 percent greater when S-4 was removed. Conversely, FE results indicated that the loads at the base of the excavation would be about half of that estimated with the apparent pressure diagrams for the final wall configuration. The primary reason for this difference is that the FEA is able to capture the influence of the hard clayey silt layer at the base of the excavation that acts as an "in situ" strut, thereby limiting wall deformation and attracting horizontal loads.

Table IIb shows the effect of installing tieback anchors at the first 3 levels in lieu of internal struts. The wale loads are comparable to the previous estimates at the first two anchor levels. In the final configuration, the third anchor level carries loads close to the apparent pressure diagram. In a well-designed braced wall system in which soil strengths are not fully mobilized, the anchors tend to carry the design preload. The loads on the bottom strut levels are higher when anchors are used at the top three levels instead of internal struts. The relatively stiff lower two internal struts appear to attract more load when the top three supports are anchors. The bottom wale load is still significantly lower than that estimated with the apparent pressure diagrams because the FEA captures the effect of the hard clayey silt layer at the base of the excavation.

Table IIa. Estimated Wale Loads Using Cross-Struts

SUPPORT LEVEL	PRELOAD (k/ft)	APPARENT PRESSURE DIAGRAM (k/ft)		FE RESULTS (k/ft)	
		with S4 (3a)	S4 removed (3b)	with S4 (4a)	S4 removed (4b)
(1)	(2)				
S1	10	22	22	20	20
S2	20	22	22	21	21
S3	20	20	39	41	50
S4	10	32	--	28	--
S5	15	45	58	15	26

Table IIb. Estimated Wale Loads Using Soil Anchors

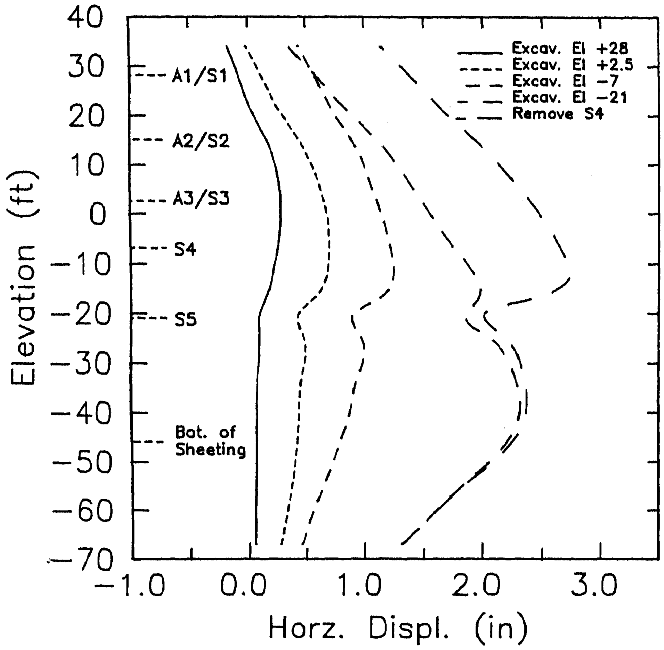
SUPPORT LEVEL	PRELOAD (k/ft)	FE RESULTS (k/ft)	
		with S4 (3a)	S4 removed (3b)
(1)	(2)		
A1	20	20	20
A2	20	20	20
A3	36	37	39
S4	10	35	--
S5	15	15	37

Wall Movements

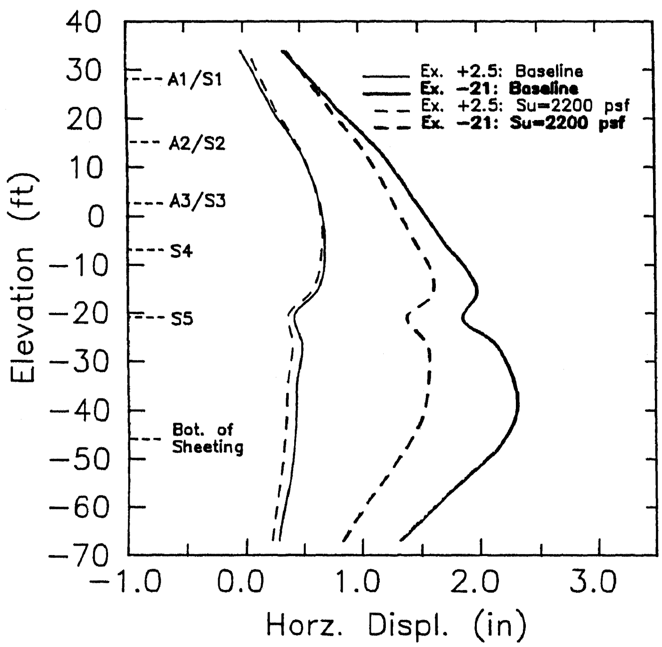
The characteristics of the wall support components and the wall itself are important aspects in limiting wall and hence surface settlement (Clough and O'Rourke 1991). A number of wall configurations were analyzed to investigate the sensitivity of ground deformations to the bracing system employed in the reconstruction scheme. Because of project time constraints, flexibility was limited to exploring such matters as struts vs. anchors, numbers of anchors, and magnitude of preload. The primary objective of the FEA in this study was to estimate the likely range of ground movements and to assess the sensitivity of the wall performance to potential design modifications and to reasonable variations in the subsurface conditions.

The FE prediction of the lateral earth movement behind the sheet pile wall at the critical absorber building (inclinometer 8A, Fig. 1) is shown in Figure 6(a). At this location, anchors were installed at the top three bracing levels. The restraining effect of the hard clayey silt layer at the base of the excavation (Elev. -21 feet) is readily apparent. Before removal of temporary strut S4, the maximum lateral wall deformation occurs below the base of the excavation at Elev. -40 feet. The FE results suggest that significant movement would occur at the S4 level when S4 was removed. These results emphasize the importance of the temporary strut S4 and the need for a stiff strut at S5. The 2-D baseline FE model indicated maximum lateral wall deformations of 2 to 2½ inches. The maximum ground surface settlement calculated by all FEA was typically three quarters of the maximum lateral wall movement. Hence, these 2-D FE results indicated that unless the excavation length was restricted, it would be difficult to keep the surface settlements below that desired on this project (< 1 inch).

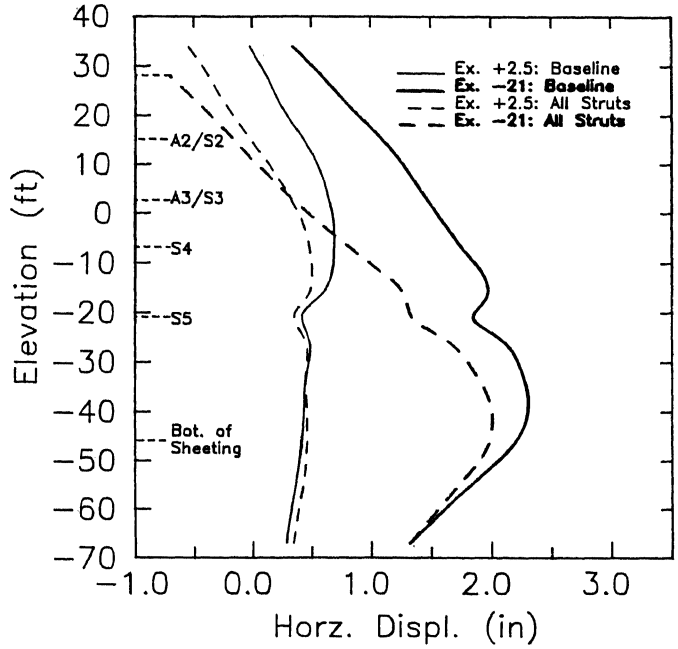
Figures 6(b to d) illustrate the sensitivity of the lateral movements to a number of proposed design modifications and reasonable variations in the subsurface conditions. In these figures, the wall deformation is shown for two stages of excavation: (A) Excavation to Elev. +2.5 ft. and installation of S3/A3 and (B) Excavation to Elev. -21 ft. before removal of S4. Figure 6(b) illustrates the importance of assessing the undrained shear strength of the base clay when the Factor of Safety against base heave is low. In this case, increasing the undrained shear strength of the base clay by 20 percent reduced the maximum lateral wall movements by 30 percent. Unfortunately, on



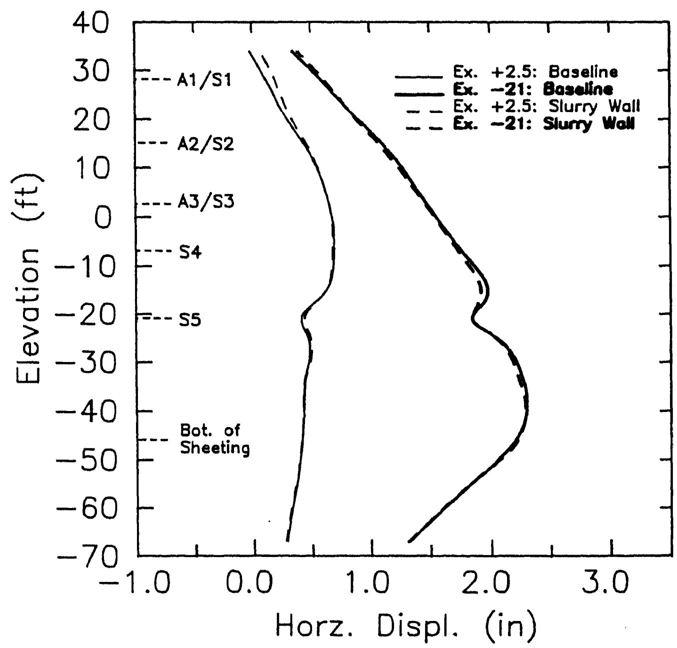
(a) Baseline FE Estimates of Lateral Ground Movements at Inclinometer 8A



(b) Effect of Increasing Base Clay (layer 5) Su from 1800 psf to 2200 psf



(c) Effect of Using Internal Struts Instead of Anchors at the Upper Three Levels



(d) Effect of Using Slurry Wall Instead of Heavy Sheet Piling

Fig. 6 Results of Preliminary FE Sensitivity Study this fast-track project, reliable strength tests on the base clay could not be completed within the time allowed as the retrieved soil samples were disturbed. If time permits, much could be gained by characterizing the strength and stress history of the base clays.

The FE results (Figure 6(c)) indicate that the use of internal struts was more effective in reducing lateral movements than the use of ground anchors at the first 3 levels. The development of a plastic zone below the third anchor (A3) in the upper medium stiff clay (layer 3) is primarily responsible for this additional movement. Anchors were beneficial because of the unrestricted excavation, but there was concern that creep would reduce the anchor load and lead to excessive ground deformation. Consequently, anchors were periodically fitted with load cells at the third layer. In some cases, the anchors did not hold their load, and supplemental struts were installed. In the absence of FEA, load cells might not have been installed, hence the need for supplementary struts to avoid excessive subsidence would not have been perceived in time.

The PZ 40 sheet pile wall with roughly 10 foot vertical strut/anchor spacing (h) produced a stiff bracing system. Using the Mana and Clough (1981) stiffness factor $(EI)/(\gamma_w h^4)$, the system's stiffness was intermediate between that of typical sheet pile walls and stiff slurry walls. The FE results (Fig. 6(d)) indicated that there would be virtually no reduction in lateral wall movements if a 3-foot thick slurry wall was used in lieu of the steel sheet piling. Finally, other FE results indicated that prestressing the struts was effective in significantly reducing lateral wall movements.

In summary, the preliminary FE results emphasized these key factors regarding minimizing ground movements:

- require excellent workmanship (eg. follow the planned excavation procedure, quickly install bracing and prestress, and use tight steel shims)
- capitalize on 3-D effects by minimizing the open excavation length
- preload struts and anchors
- capitalize on the hard clayey silt layer at the base of the excavation which acts as an in situ strut
- the most critical subsurface condition is the undrained shear strength of the base clay (lower Su = lower FS_{BH} = larger movements)
- yielding of the medium stiff clay below the A3 anchorage zone could produce excessive movements

Consideration of these factors led to the development of the estimates of maximum lateral wall movements presented in Table III. The first two estimates based on past observations (Clough et al. 1989) indicated maximum wall movements of between 3-4 and 6-8 inches might be expected. The third estimate is based on the observed performance of bracing systems in more favorable soil conditions and hence represent an approximate lower bound 2-D estimate (1-2½ inches). The 2-D FEA estimated maximum

TABLE III. Estimated Maximum Lateral Wall Movement

Description (1)	$\delta_{H\,max}/H$ (%) (2)	$\delta_{H\,max}$ (in) (3)
Soft to Medium Stiff Clay, Base Heave FS ~ 1.2 (Clough et al. 1989)	~1.0	6-8
Soft to Medium Stiff Clay, Base Heave FS ~ 1.8 (Clough et al. 1989)	~0.5	3-4
Stiff Clays, Residual Soils and Sands (Clough and O'Rourke 1991)	~0.3	1-2½
2-D FEA with SOILSTRUT Sand Overlying Medium to Very Stiff Clay	~0.3	1½-2½
FEA with 3-D Effects Sand Overlying Medium to Very Stiff Clay	~0.2	1-1¾

wall movements on the order of 1½ to 2½ inches. Incorporating 3-D effects, our preliminary "best estimate" was 1 to 1¾ inches, assuming excellent workmanship on the part of the contractors. Thus, the FE analysis indicated that the surface settlements could be kept close to the desired magnitude of 1 inch.

INTEGRATION OF FEA RESULTS IN THE CONSTRUCTION PROCEDURES

Initially, the braced excavation design called for 5 levels of internal pipe struts with 20 foot spacing. Design changes implemented with use of the FEA were:

- Restrict open length of excavation at base to roughly 60 ft.
- Excavation could be opened down to the second bracing level along the entire length of excavation
- Preload struts
- The three upper struts could be replaced with anchors
- Install load cells to monitor creep in the level 3 anchors
- Add internal pipe strut 10 ft. o.c. at the base of excavation

With preliminary FEA results indicating reduced ground surface settlement when considering 3-D soil surcharge effects, construction was scheduled to minimize the open length of the trench excavation at Elev. - 21 ft. With the pipeline sections purchased in 40 foot lengths and the requirement for a minimum of two pipe cross-struts to be in place at the temporary fourth bracing level, the minimum length of the open excavation at this elevation was 65 feet. Accordingly, the following sequence for the installation of the bracing members was developed (see Fig. 7):

(1) The FE results indicated that removing the soil for the placement of the top two levels of bracing resulted in minimal ground surface settlements. Hence, the bracing at these levels

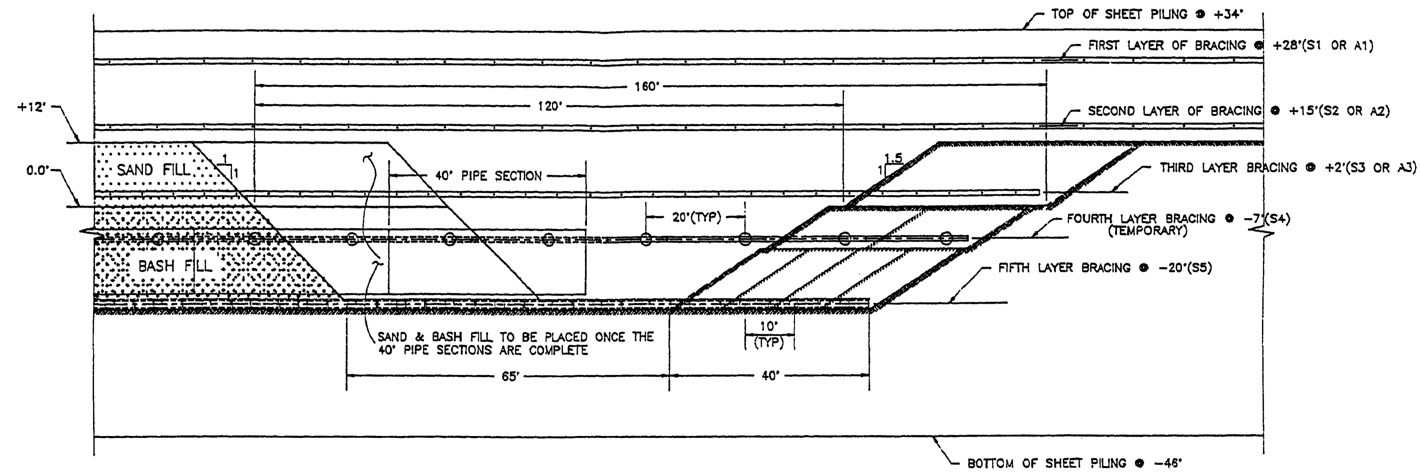


Fig. 7 Typical Construction Sequence for each 40-ft. Section of Pipe Installation

were installed without restrictions in the length of excavation that could be opened up at one time.

(2) Excavation for placement of the third bracing level significantly reduced the benefits of 3-D effects. Hence, the third layer of bracing was installed in sequence with the placement of the new pipelines. Placement and welding of the pipelines took four days, hence, soil anchors at the 3rd level, were placed in 40-foot panels, which also required four days for installation and testing.

(3) The fourth bracing layer consisted of pipe cross-struts and walers placed in 20-foot panel lengths. To restrict the length of open excavation to 100 feet at this level, the placement of the pipe struts and the walers at one end of the excavation could not start until a pipe section was installed and welded, and backfill was ready for placement at the other end.

(4) FE results indicated that it was advantageous to stiffen the bottom support level before the concrete set, hence, 24 inch diam. pipe struts were installed at the base of the excavation and embedded in concrete. The bottom bracing layer consisted of pipe cross-struts placed on 10-foot centers. Once four sets of cross-struts and walers were in place, a 2-ft. thick concrete slab, which enveloped the bracing members, was placed.

As construction proceeded and the wall's performance was monitored, a number of revisions were implemented to reduce delays:

(1) To minimize delays in pipe placement caused by excavation and installation of bracing at the fourth and fifth level, the construction sequence was modified such that the excavation of the next segment was permitted when the previous pipe section was in-place but prior to welding.

(2) Delays also occurred when a third level soil anchor failed to hold its design load. Insufficient time was available for reinstallation of anchors due to the grout cure time. This problem was resolved by allowing the excavation for the third layer to proceed unrestricted once the braced wall was beyond the structures sensitive to ground movements. Construction monitoring had indicated that the observed wall and ground deformations were less than initially estimated. However, the wall deformation was higher in the region where excavation was allowed to proceed unrestricted to the third layer, demonstrating the importance of this restriction in sections adjacent to sensitive structures.

A view of the excavation support system in the vicinity of the main power plant is shown in Figure 8. A transition from internal struts to anchors at the upper three levels is shown in this figure. It was decided to leave the sheeting and all of the bracing members in-place (except for the fourth layer) to eliminate movements associated with the removal of the bracing members.

CONSTRUCTION MONITORING

An extensive instrumentation program was undertaken to assess wall stability and monitor ground movements. Site instrumentation included piezometers, inclinometers, surface monuments, tiltmeters and electrolevels on structures, strain gauges on struts and load cells on the tieback anchors. Figure 1 shows the location of the inclinometers.

Fig. 8 Excavation and Installation of Retaining System

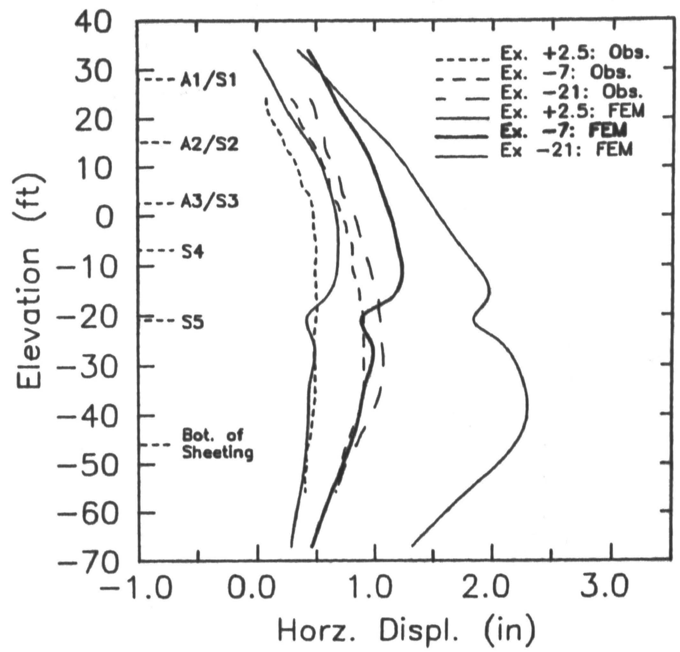


Fig. 9 Comparison of FE Results and Inclinometer 8A Measurements

One of the primary objectives of the instrumentation program was to provide a detailed indication of construction site behavior on a real-time basis. The instrumentation program, however, was designed and implemented independently of the contractor and his consultants. Key aspects of the equipment installation and data reduction were not coordinated with the persons who would be interpreting the results. For this reason some instruments were not installed optimally, and often the interpretation of the measurements was not unambiguous. For example, only two inclinometers were sited close enough to the excavation to allow calibration of the FE model during construction. In addition, the bottoms of the casing were founded in zones that exhibited significant movement. Interpretation of the inclinometer data required offset surveys of the top of casings and judgment in conjunction with the FEA because there was no point of fixity. Measurements made on the surface movements, electrolevels, tiltmeters and field observations, however, provided additional confidence in the interpretation of inclinometer measurements.

The measured lateral ground movements at the critical absorber structure (Inclinometer 8A) are qualitatively compared to the baseline 2-D FE results in Figure 9. The 2-D FE results provide a fair prediction of the observed wall movements initially (i.e. excavation to Elev. +2.5 ft. and installation of A3) but greatly overpredict ground movements at later excavation stages. Since the 2-D FEA does not capture the 3-D effect of restricting the length of excavation open at the bottom, the 2-D FE results should, and do, overpredict lateral wall movements during the later stages of excavation. The pattern of lateral earth movements behind the sheet piling is fairly similar, although the FE model's discretization overemphasizes the reduction in lateral movements in the vicinity of the hard clayey silt layer. The observed maximum lateral wall movement adjacent to the absorber building of 1 to 1¼ inches was in good agreement with our initial "best estimate" of 1 to 1¾ inches (see Table III).

Figure 10 depicts the observed movements at Inclinometer 12A. The excavation constraints were relaxed slightly at this location to reduce construction time. This was allowed since sensitive structures were not adjacent to the excavation in this area and previous observations of ground movements were in general agreement with the initial estimates. Lateral ground movements were larger at Inclinometer 12A than at Inclinometer 8A. Two factors contribute to this observation. First, near Inclinometer 12A, excavation was permitted down to the third level, not the second level, before restricting the length of the open excavation. Second, the third level anchors (A3) (at Elev. +2.5 ft.) were not installed and prestressed as timely in this region. Significant lateral movements occurred at this level due to a six day delay in prestressing A3 anchors. These observations emphasize the importance of the 3-D effects and the tight excavation control in reducing ground movements during this project.

Finally, the FEA provided reliable estimates of the measured anchor loads. In general, the anchor loads predicted by the SOILSTRUCT FEA were within 15 percent of those measured at the third anchor level during the project.

CONCLUSIONS

Conventional analyses estimated that the factor of safety against base heave for this 60 foot deep excavation was close to one. NIPSCO man-

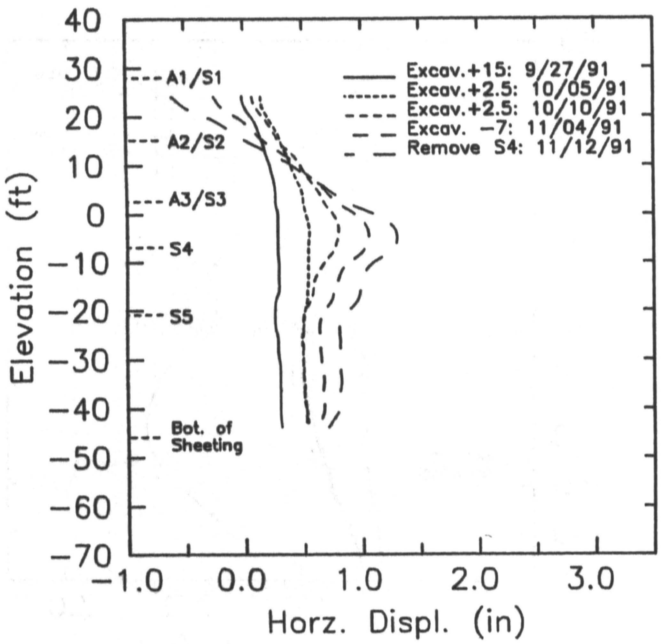


Fig. 10 Inclinometer 12A Measurements

agers, however, required that the power plant become operational as soon as possible without adversely affecting appurtenant structures. The established tolerable surface settlements were stringent (≤ 1 inch). Nevertheless, it was considered that expeditious, but safe, construction operations would be possible through the integration of FE analysis and construction monitoring.

The efficiency of the braced excavation system constructed at the Bailly Generating Station was significantly enhanced by incorporating FEA results and construction monitoring data. Modifications made to the preliminary design included replacing the three upper internal cross-struts with soil anchors to speed up construction operations; allowing excavation to the depth of the second anchor level over the full length of the excavation; installing load cells on the third anchor level to monitor stress relaxation and the potential for additional ground movements; and installing pipe struts 10 ft. o.c. at the fifth bracing level. The open dialogue between the construction contractor and the engineering consultant (on nearly a daily basis) regarding the excavation sequence, design modifications and planned contingencies was a crucial aspect of the success of this project. The NIPSCO Bailly Station Power Plant was in operation just 150 days after the original intake and discharge pipelines collapsed. The "better-than-expected" performance of the flexible retaining structure was largely due to the tight controls on the excavation procedure which included restricting the length of excavation to take advantage of 3-D effects combined with a high level of construction quality control.

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