Permanent Tieback Retention System

T. C. Anderson
Schnabel Foundation Company

M. E. Lockwood
The H. C. Nutting Company

M. F. Nethero
The H. C. Nutting Company

Permanent Tieback Retention System

T. C. Anderson, P.E.
Design and Construction Manager, Schnabel Foundation Company

M. E. Lockwood, P.E.
Geotechnical Engineer, The H. C. Nutting Company

M. F. Nethero, P.E.
Vice President, The H. C. Nutting Company

SYNOPSIS This paper describes the design, construction, and monitoring of a permanent tieback retention system which permitted a 55-foot-deep excavation for an 16-story addition to the existing Good Samaritan Hospital in Cincinnati, Ohio. Tieback anchor capacity is developed in moderate-to-low-strength shale bedrock with intermittent thin limestone layers. The retention system provides temporary and permanent support for adjacent 5- and 10-story buildings and unbalanced lateral earth pressures due to sloping site topography. A permanently tiedback wall also supports a 17-to-33-foot-deep cut adjacent to the 8-story parking structure in lieu of a conventional retaining wall.

INTRODUCTION

The permanent tieback retention system for this project involved the installation of drilled-in soldier piles and tiebacks. The retention system design required temporary and permanent support of immediately adjacent 5- and 10-story buildings in an area of a 55-foot-deep excavation and permanent restraint of unbalanced lateral earth pressures developed due to sloping site topography. The tieback retention system was also the most cost-effective approach for temporary and permanent support of an adjacent 8-story parking garage where a 17-to-33-foot-deep cut was required.

The tieback anchor capacity was developed in moderate-to-low-strength shale. For this reason, field tests were performed on each tieback to identify the load-deformation behavior and to provide data to enable a decision to be made as to their adequacy. The creep rate exhibited by these tests was extrapolated to predict the long-term performance of the tieback retention system. Instrumentation monitoring has been performed to verify satisfactory long-term retention-system performance consistent with design expectations such that tieback anchors undergo no excessive movement or loss of capacity.

PROJECT DESCRIPTION

The 16-story hospital addition was situated within a congested structure area, such that six existing buildings bordered the addition (see Figure 1). Surface grades, prior to construction, varied in elevation as much as 40 to 45 feet falling southeast to northwest. The slab-on-grade elevation of the addition was 15 to 55 feet below preconstruction grades.

The subsurface investigation performed within the area of development defined the soil and bedrock profile and established material properties necessary for design parameter selection. The subsurface investigation indicated that existing cohesive fill mantled most of the site generally varying between 3 and 7 feet in depth. A maximum fill depth of 17 feet was encountered within the south central portion of the site. Moderately plastic residual overburden, developed from the underlying bedrock, was situated beneath the surface fill and contained increasing fragments and floater slabs of limestone with increasing depth below grade. This overburden had a stiff consistency and a maximum thickness of 10 feet. Horizontally bedded, layered brown and gray Ordovician Age shale, with intermittent thin fossiliferous limestone layers, lies immediately below the fill in most areas adjacent to existing structures due to historic grading changes. Brown weathered shale was predominant within the upper 10 feet of rock penetration, then transitioned to essentially unweathered gray shale below. The upper bedrock contained approximately 50 percent limestone and decreased to only 15 percent limestone in the lower elevations. The bedrock formation within the lower elevations was known to deteriorate readily when exposed to the elements and was associated with landslide-prone topography in the local area. The subsurface investigation

Figure 1. Aerial View of Project During Early Construction Phase

also indicated that seepage areas were common at the limestone-shale contacts, producing weathered zones throughout the bedrock profile. Laboratory tests disclosed bedrock properties having the following average characteristics: 144 pounds per cubic feet dry density, six percent moisture content, and 25 tons per square foot shear strength.

Within the existing hospital area, the depth of excavation was a maximum of 55 feet (see Figure 2). Additionally, 200-to-648-kip-column loads from immediately adjacent 5- and 10-story buildings imposed surcharge forces on the retention system.

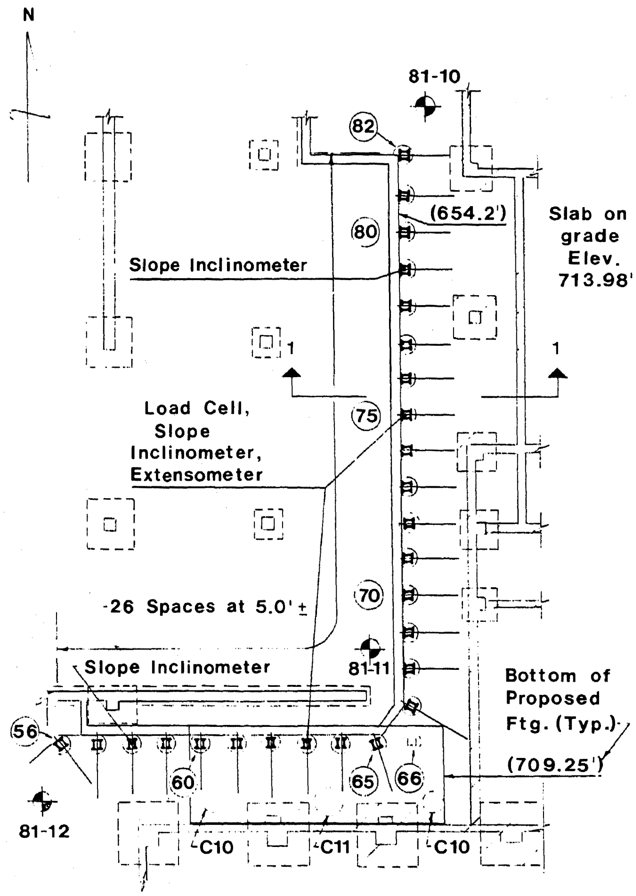

Figure 2. Plan for Existing Hospital Area

Due to the earth and surcharge pressures, a permanent tieback retention system was selected for design. The new structure was judged as having to accept up to 50 percent of the total design pressure. Each below-grade floor slab was designed to key into the drilled piers of the retention system. Therefore, it was essential that the tiebacks not undergo excessive movement or loss of capacity during the life of the structure in order to satisfy design criteria.

In the parking structure area (see Figure 3), a 17-to-33-foot-deep cut was required for the access drive. Rather than constructing a conventional retaining wall requiring tempora support during construction, a permane tieback retention system with a cast-in-pla wall facing was selected, due to the cos effectiveness and site and space contraint Of primary importance was the limitation lateral deflection utilizing permanent tieback

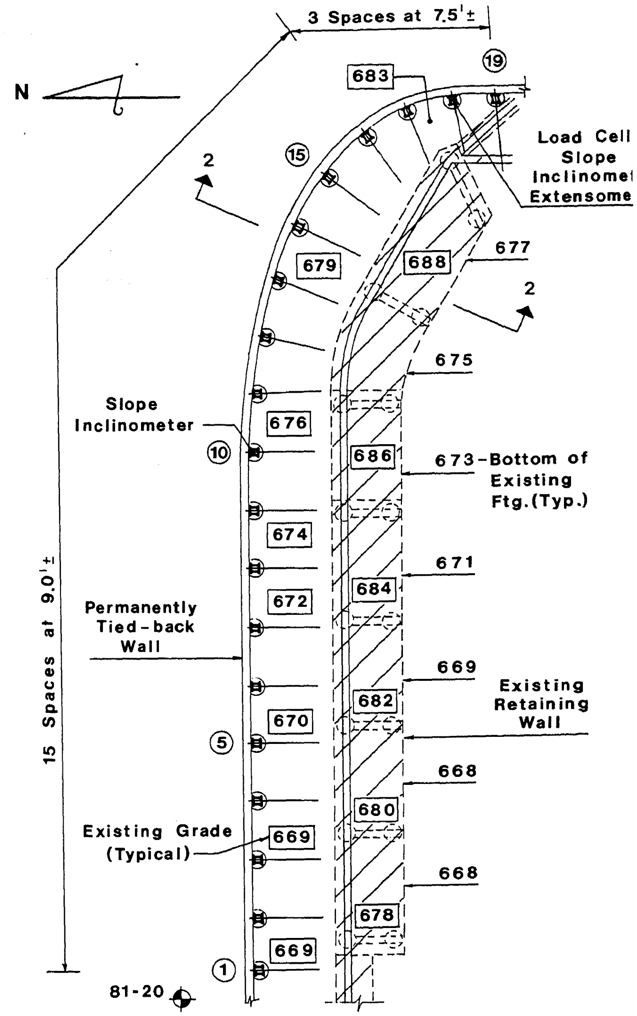

Figure 3. Plan for Parking Structure Area

DESIGN

For the design of the retention system, a lim equilibrium trial wedge analysis was utilize assuming the rock behaves as a soil mass wi a friction angle of 35 degrees. This analys incorporated the effects of the adjacent fou dation loads. Once the maximum stabilizi force (P_A) was determined from the resulti force polygon, this force was divided by t height of excavation and distributed on the ba of the wall as a uniform pressure.

Figure 4 shows the forces acting on the tri wedge, as well as a typical force polygo The resulting design pressure diagrams f Sections 1-1 and 2-2 are shown in Figures and 6, respectively.

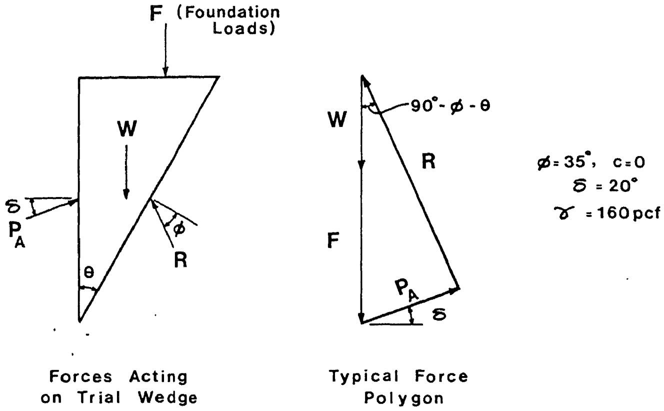

Figure 4. Limit Equilibrium Trial Wedge Analysis

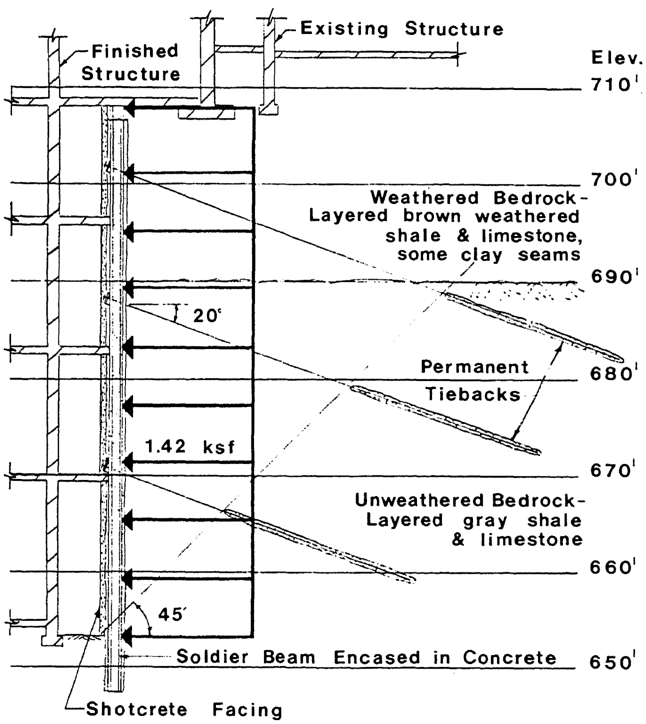

Figure 5. Section 1-1

The soldier piles for both walls consisted of a pair of wide-flange beams strapped together with a 5-inch-wide space between the beams. The soldier piles were typically set in 30-inch-diameter predrilled holes backfilled with 3000 psi concrete. At the existing hospital area, the soldier pile spacing was 5 feet center-to-center with lateral support provided by three to four levels of tiebacks. For the parking structure area, the soldier piles were spaced at 9 feet on center and supported by one to three tiers of tiebacks. The design relied upon the arching effect between the soldier piles permitting an economical pile spacing. Timber lagging between soldier piles was required only in areas of existing fill or overburden.

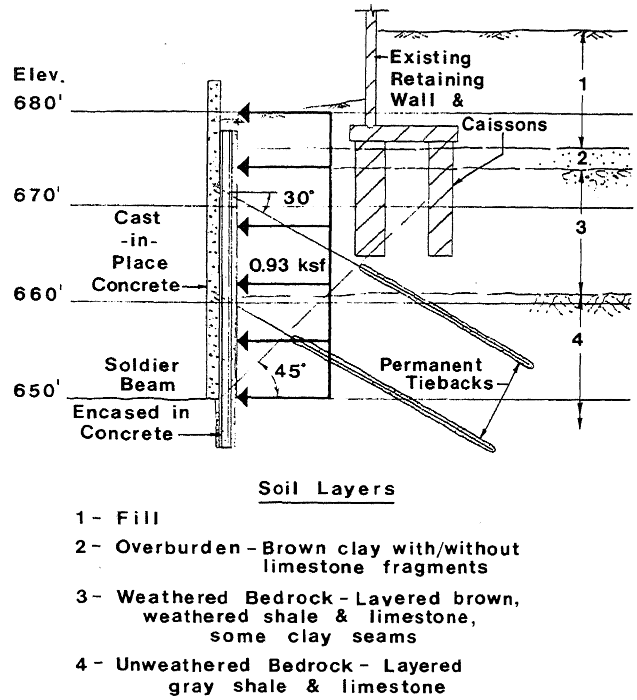

Figure 6. Section 2-2

Tieback design loads generally varied from 80 to 140 kips, which resulted in the use of 1-1/4-inch- and 1-3/8-inch-diameter Dywidag thread bars of 150 ksi ultimate strength. As indicated in Figures 5 and 6, the tiebacks were installed at 20 to 30 degrees below the horizontal. The anchor lengths varied from 20 to 30 feet with a minimum anchor diamater of 3-1/2 inches. In order to prevent the possibility of failure or movements extending behind the tieback anchors, anchorage behind a 45-degree plane rising from the base of the excavation was provided even though the actual critical surface was significantly steeper.

CONSTRUCTION

The holes for the soldier piles were drilled with a crawler-mounted drilling machine using rock augers and core barrels. However, due to the hardness of the limestone layers in the unweathered bedrock, coring was an extremely slow operation. For this reason, a 30-inch-diameter down-the-hole hammer was utilized to drill the harder rock. This hammer worked fairly effectively even though the soft shale between the limestone layers tended to clog the ports in the hammer bit.

A large Gardner-Denver air track drill was employed to percussive-drill the tieback holes through the 5-inch space between the wide-flange beams of the soldier piles. No water was introduced into the tieback holes in the drilling process. After a 4-1/2-inch-diameter hole was advanced to the intended depth, a grout tube was used to tremie-grout the hole from the bottom until clean grout emerged at the surface. Finally, the tendon was inserted into the grout-filled hole.

For the existing hospital-area wall, a 6-inch-thick shotcrete facing with one layer of wire mesh was placed against the cut face and anchored to the piers comprising the soldier piles. Wall drainage behind the shotcrete facing consisted of a 2-inch minimum thickness of porous filter material against the rock face between the piers held in place by diamond mesh and visqueen anchored to the piers. The final wall facing for the parking structure area consisted of 15-inch-thick reinforced concrete anchored to the soldier piles by shear stud connectors welded to the steel beams. Drainage behind the wall facing was provided by installing 2-inch-thick Geotech drainage board with filter fabric on the back side against the cut face. Photos taken during the construction of the retention systems for the existing hospital and parking structure areas, respectively, are shown in Figures 7 and 8.

Figure 7. Hospital Area
During Construction

Figure 8. Parking Structure
Area During Construction

TESTING

All of the tiebacks were tested to verify tha they would carry the design load withou excessive movement. Three types of tests wer performed: performance, proof, and cree tests. For each of these tests, a calibrate hydraulic jack and pump were used to appl the load and an Ames dial gauge mounted on a independent tripod was utilized to measur the movement of the tendon to the neares 0.001 inch.

Two of the initial tiebacks installed in th upper and lower rock units were creep-tested During the creep performance test, the tiebac was incrementally loaded and unloaded up to maximum of 133 percent of the design load Each load increment was held constant usin an electrical resistance load cell for 10 t 60 minutes, with the exception of the fina load which was held for 24 hours, and th elongations recorded. Figure 9 shows the plo of one of the creep performance tests.

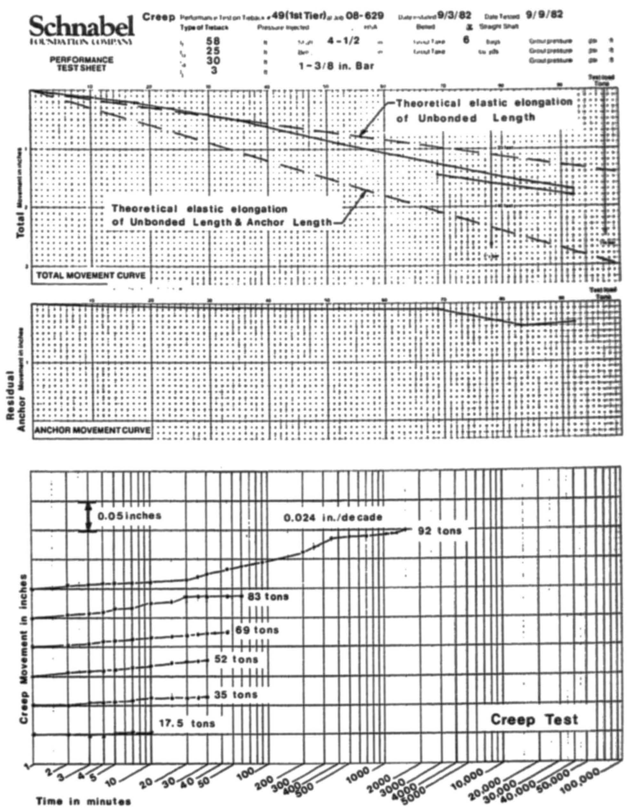

Figure 9. Creep Performance Test

The upper graph in this Figure shows the tota tieback movement as a function of load, whil the middle graph shows the residual movement o the anchor as a function of load. The residua movement (permanent set) of the anchor is th non-elastic or unrecoverable movement of th anchor which is measured when the load i released after each loading increment.

lower graph of Figure 9 shows the plot of p movement versus time on a semi-logarithmic h, with each curve representing the creep ment at each load increment. The criteria acceptance was that the creep movement s had to be approximately straight lines or ave downward. Both creep tests indicated p rates of 0.024 inch per log cycle, which d produce a creep movement of approximately inch over a period of 50 years.

hly five percent of the remaining tiebacks performance-tested. The performance test the same incremental loading and unloading edure as the creep test, except that only maximum load was held constant. The tie- was considered acceptable if the movement ng the 10-minute load-hold was less than inch, otherwise the load had to be main- ed for 60 minutes so that a creep curve d be plotted.

remaining tiebacks were proof-tested by uring the load applied to the tieback and movement during incremental loading to a mum of 133 percent of the design load. The mum load applied during the proof test was constant for 5 minutes and the tieback ment recorded. If the movement during the nute observation period was less than inch, the test was discontinued. If the ment exceeded 0.01 inch, the load was main- ed until a creep rate could be determined compared to the creep behavior observed ng the performance or creep tests.

the 191 permanent tiebacks installed on project, the failure rate using the above ing procedures was approximately 5 percent.

ACK CORROSION PROTECTION

tieback tendons were protected against osion by using Schnabel Foundation Company's nted corrosion-protection system. This sys- is shown in Figure 10 and consists of a ination of an electrostatically applied y coating on the Dywidag bars and a heat- inkable polyethylene tube internally coated a thixotropic sealant. In the anchor gth, protection is provided by the epoxy ing and the cement grout around the tendon. tralizers were utilized in the anchor length maintain a minimum 0.5-inch grout cover. In unbonded length, the heat-shrinkable tube installed over the epoxy-coated tendon provide the required level of corrosion tection.

critical area of the tendon below the ring plate was protected by a PVC trumpet led with an anti-corrosive grease as illus- ted in Figure 10. In order to interrupt ential long-line differential aeration and ay-current corrosion systems, electrical lation was provided by an insulation pad ow the bearing plate.

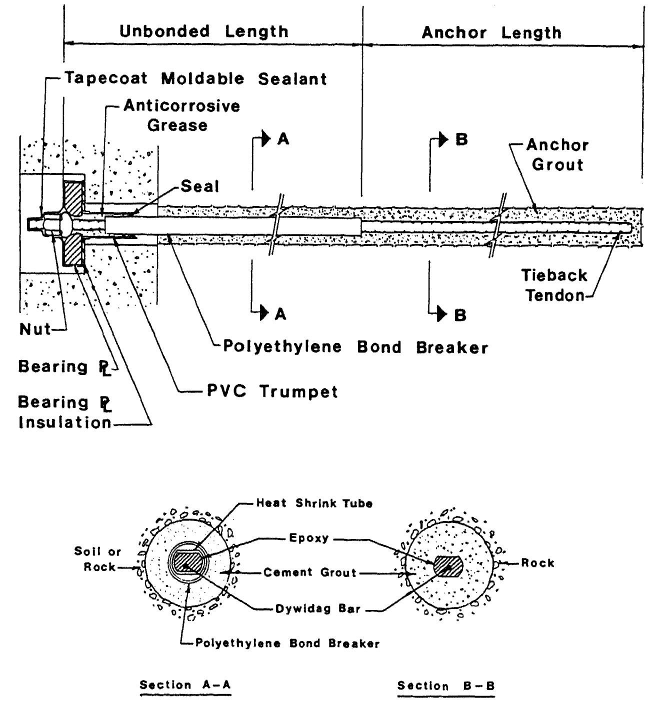

Figure 10. Insulated Simple Corrosion-protected Tieback

MONITORING

The immediate and long-term performance of the permanent tiebacks has been monitored through a combination of several instrumentation methods. These methods included the use of digitilt inclinometers, extensometers, load cells, and optical survey for both lateral movement and settlement of adjacent structures. The monitoring, on a short-term basis, provided a check on the tieback performance as the excavation proceeded and the resulting influence on adjacent existing structures. Monitoring on a long-term basis provided a continual means of evaluating the performance of the tiebacks during the life of the structure. Long-term monitoring instrumentation was selectively placed establishing six vertical sections along the retention system. Data from each method was correlated with the other methods. Instrumentation was established at four locations in which the maximum excavation depth occurred near adjacent existing hospital structures (see Figure 2), and two vertical sections of the permanently tiedback wall along the parking structure (see Figure 3).

Optical survey monitoring was performed by an independent survey team with readings taken at weekly intervals. This optical survey determined lateral deflections of each soldier pile within the retention system, as well as lateral deflection and vertical settlement of adjacent structures. Initial readings were made prior to the beginning of excavation.

Monitoring readings for digitilt inclinometers, extensometers, and load cells were taken consistent with excavation and construction schedule. Initial readings for the digitilt inclinometers were made prior to beginning excavation. Initial readings for the extensometers and load cells were made at the time of installation. The digitilt inclinometer casing was installed within selected drilled-in soldier piles during concrete placement. This casing extended from the bottom of the piles to several feet above the top of the piles. During the excavation process, monitoring readings were taken prior to the installation of each row of tiebacks and at the time the excavation reached the final elevation. Beyond this point, monitoring readings were taken at 2-week intervals until the structure was completed to one level above exterior grade. Readings were scheduled at 6-month intervals until the structure was completed and at 1-year intervals for a 5-year period beyond structure completion.

Inclinometer casing was installed in six soldier piles within the retention system and readings performed with a SINCO Model 50325 Digitilt Inclinometer. A total of six IRAD GAGE Type H-300 Load Cells were installed at the first and third tieback levels at three soldier pile locations which also included the digitilt inclinometer instrumentation. Three rod-type extensometers were installed at the first tieback level at three soldier pile locations which also included load cell and digitilt inclinometer instrumentation. A depth micrometer was utilized to measure travel of the rod relative to the sleeved fitting within the soldier pile.

Figure 11 provides a graphical plot of instrumentation monitoring data at pile no. 63 within the retention system. Instrumentation at this section included digitilt inclinometer, extensometer, and load cells. The data plot for November 1982 represents conditions in which all tiebacks had been installed and the excavation had reached final elevation. The data plot for November 1983 depicts lateral deformation and change in anchor capacity which has occurred in a 1-year time period. The plotted deformation associated with load cell readings was derived from the theoretical strain elongation of the unbonded tieback length associated with the change in anchor force for the 1-year period. There is excellent correlation between the instrumentation monitoring methods. Based on the data, lateral deflections for the 1-year period typically were between 0.04 inch and 0.08 inch. Based on load deformation plots developed from the creep tests performed on project tiebacks, 0.05 inch to 0.08 inch of tieback deformation was predicted for the first year of tieback performance.

Instrumentation monitoring at the other test sections indicated similar lateral deflection over the first year of tieback performance. Maximum deformation at each test section typically occurred near the top of the retention system with values varying between 0.05 inch and 0.17 inch. Generally, lateral deflection between the upper tieback level and the maximum depth of excavation over the first year was less than 0.10 inch. Most of the lateral deflection for each test section occurred during the excavation process.

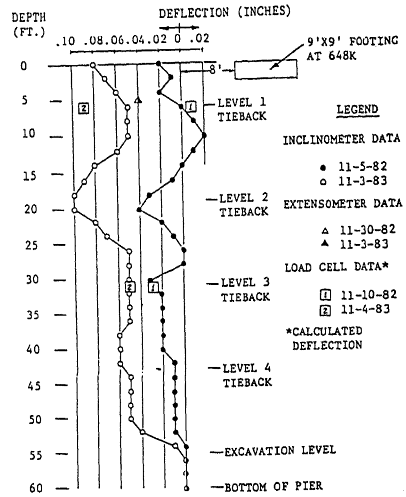

Figure 11. Graphical Plot of Instrumentation Monitoring Data

CONCLUSIONS

Due to cost-effectiveness and site and spac constraints, a combination temporary and per manent tieback retention system was selected t support 17-to-55-foot cuts adjacent to existin structures. The retention system for thi project dictated that tieback anchorage b developed in moderate-to-low-strength shal bedrock. The performance of tieback testing a the time of installation verified that projec tiebacks would carry the required loads withou excessive movement. Data from these tiebac tests enabled prediction of tieback behavio on a long-term basis. The instrumentatio monitoring has provided verification of th short- and long-term integrity of the permanen tieback retention system. Furthermore, th data developed during instrumentation monitorin has revealed that actual system performance ha been in excellent agreement with the latera deformations predicted, based on data develope during the tieback tests. The permanent tiebac retention system constructed for this projec has performed consistent with design expecta tions and has affirmed the reliability o permanent tiebacks developing anchor capacit within moderate-to-low-strength shale bedrock.

ACKNOWLEDGEMENTS

The writers wish to acknowledge the followin list of organizations and associated personne involved with this project:

Owner:	Good Samaritan Hospital Cincinnati, Ohio
Architect:	Dalton, van Dijk, Johnson & Partners Cleveland, Ohio
Structural Engineer:	THP Limited Cincinnati, Ohio
Geotechnical Engineer:	The H. C. Nutting Compan Cincinnati, Ohio
Construction Manager & General Contractor:	Turner Construction Co. Cincinnati, Ohio
Subcontractor for Retention System:	Schnabel Foundation Co. Cary, Illinois
Consultants:	Edward J. Cording David E. Weatherby