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Design and Field Monitoring of 70-Foot High Tied Anchor Retaining Wall

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SYNOPSIS: Temporary tied anchor retaining walls have been used extensively where deep excavations are required. However, permanent tied anchor retaining walls to provide lateral support along one side of a multi-story building are seldom utilized. The wall was monitored for deflection and tie load changes during and after construction. A partial detensioning program was instituted in order to maintain the design stresses.

INTRODUCTION

To meet site conditions and to provide for economical and flexible interior space, lateral support for a seven-story office building was provided by an integral, tied anchor retaining wall along one side. Figure 1(a) indicates the building outline in its relation to the adjacent sloping topography which necessitated permanently shoring an excavation about 800 feet in length and to a maximum depth of 70 feet. The general concept envisioned first excavating and shoring to the base elevation over the building site, constructing the building foundations, over a scheduled 6-month period, and then constructing the final exterior wall incorporating the shoring and with the tied anchor system providing the permanent lateral support of long-term earth pressures. Because of the critical, dual function of the shoring scheme, a specifically designed load cell and slope indicator installation and observation program was incorporated into the construction contract, rigorously adhered to, with data concurrently analyzed and findings acted upon.

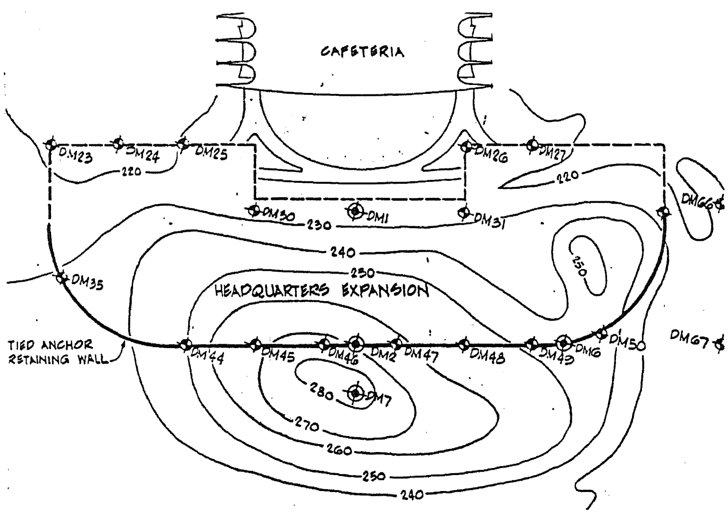

Fig. 1(a) Building Site Plan

This paper describes salient details of the subsoil conditions, the structural design and the monitoring scheme, and summarizes observations and adjustments made during construction.

SUBSOIL CONDITIONS

The building is located in the Virginia Piedmont, characterized by bedrock primarily of igneous and metamorphic origin. These are comprised of granite, shist, metasedimentary, metaigneous and metavolcanic rock types. Bedrock surface is somewhat irregular and usually covered by weathered residual material, called saprolite. Bedrock map of the site indicates gneiss and granofels, with a mineral composition of quartz, feldspars, mica and chlorites. It commonly has two steeply inclined foliations, locally faulted and sheared, with steeply dipping intersecting joints spaced 3 feet or more apart.

The specific site investigation involved drilling and recovering and evaluating samples from soil and rock borings, located as indicated on Figure 1(a).

A typical soil profile log is that for Boring DM-46, shown as Figure 2. We note soil overburden of saprolite, with texture ranging from clayey silt in the fully weathered upper stratum to the partially weathered gravelly sandy silt with increasing amounts of interbedded, moderately weathered gneissic rock as one approaches bedrock at a 75-foot depth at this location. Standard penetration blow counts N are in excess of 100 blows per 6 inches below about 30 feet from the surface, in association with the partially weathered stratum. The bedrock was cored, indicating moderately fractured granite gneiss, with about 97 percent recovery. Although the upper clayey stratum was moist, there was no free water encountered in the boring and the boring was dry upon completion.

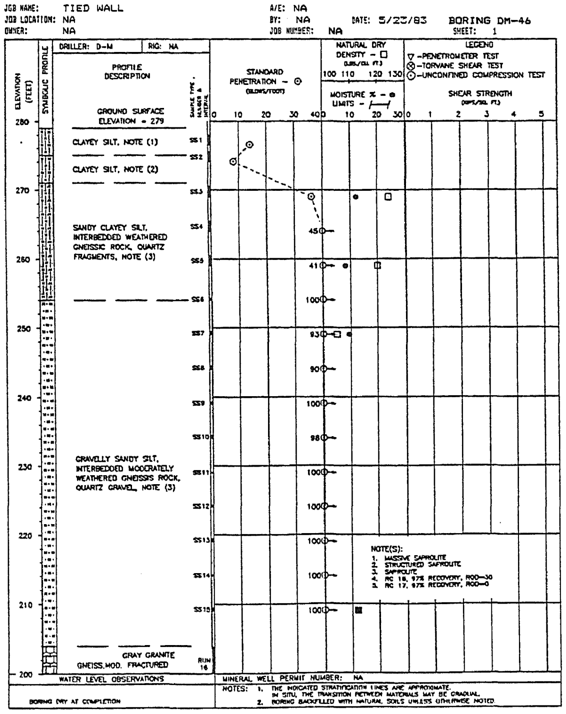

Fig. 2 Typical Soil Profile Log
Boring DM-46

A generalized soil profile is indicated on Figure 3 based on the other borings along or adjacent to the proposed location of the shored wall. The soil strata have been designated on the basis of similar, salient characteristics as follows:

Zone A - Massive saprolite, with unclear evidence of primary structural features of the parent rock. At the site it ranges only a few feet in thickness.

Zone B - Structured saprolite, with clear evidence of primary structural features of the parent rock (i.e., foliation, jointing, crystal structure). At the site it ranges generally from 20 to 30 feet thick.

Zone C - Saprolite, a transitional zone, grading from the structured, substantially fully weathered Zone B to the relatively intact bedrock (i.e., Zone D). At the site it ranges from about 40 feet in thickness at the middle half of the wall alignment to perhaps 10 to 15 feet at the ends.

It is noted that the proposed subgrade of the building roughly coincides with the bedrock surface along the wall alignment, but bedrock surface rising as much as 5 feet above subgrade Elevation 205 at some locations. This fact was recognized and accounted for in the shoring design and contract documents.

DESIGN OF PERMANENT TIE BACK SHORING AND BUILDING WALL

In view of the dual function of the tie back wall, the design lateral forces were predicated on assuring "at rest" earth reactions. The assigned pressure coefficients ranged from Ko = 0.45 (i.e., N < 60) in Zone A and upper Zone B, to a minimum 0.30 in Zone C (i.e., N > 100/6"). It was recognized that these coefficients lead to relatively conservative loadings, appreciably above those ordinarily appropriate for temporary excavation shoring. But in view of the critical nature of the structure and absence of documented observations of long-term performance of tied walls in similar circumstances, it was deemed prudent not only to assign the design lateral loads to account for the varying soil resistance, but to also incorporate a load and deflection monitoring system into specific wall elements and monitor and evaluate measurements during construction.

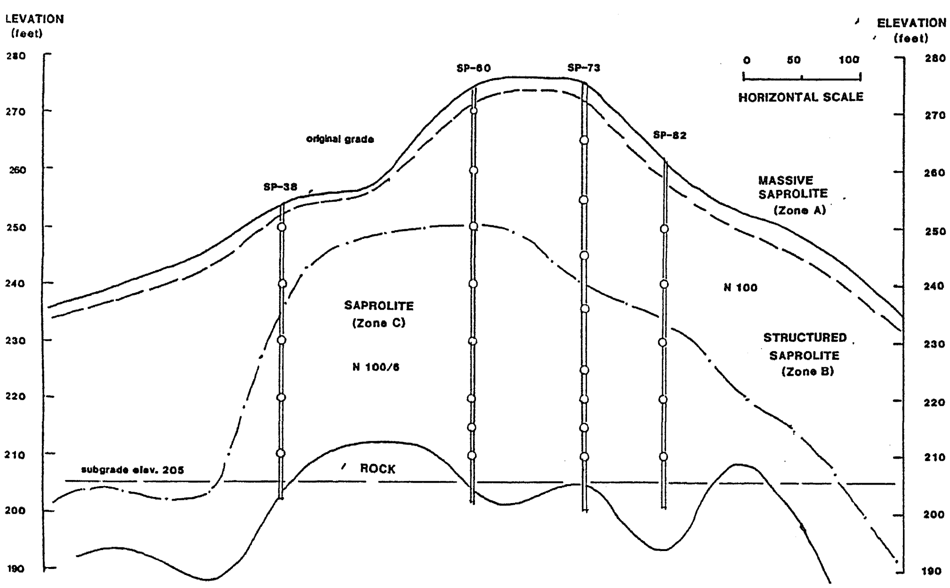

Fig. 3 Generalized Soil Profile
Location of Slope Indicators and Tied Anchor Load Cells

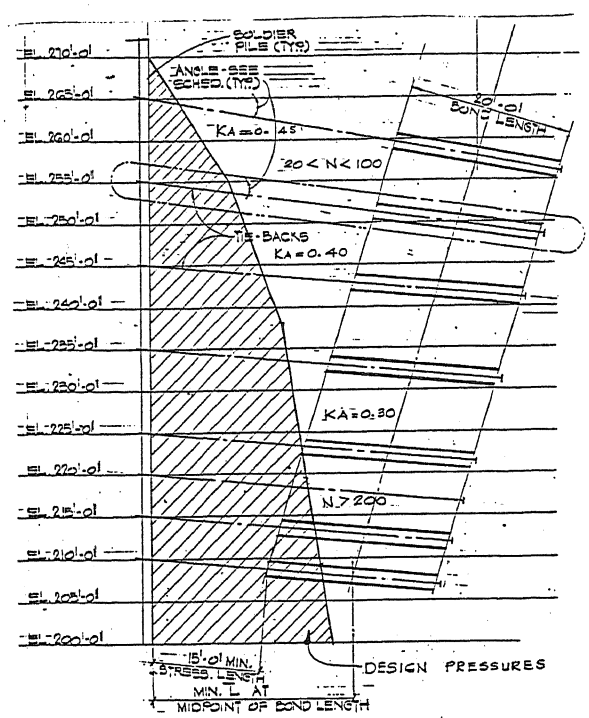

Fig. 1(b) Section thru Tied Anchor Wall including Design Pressures

Figure 1(b) shows a section through the deepest portion of the tie wall, indicating the design pressures and attendant vertical tie spacing and embedded lengths of pressure grouted anchor rods, based on soldier piles at lateral spacing of 7.5 feet to permit use of conventional wood lagging. Typical details of the soldier piles, tie anchors, and wood lagging of the temporary shoring are included in Figures 4(a) and 4(b). Details of the finished structural wall incorporating the temporary shoring and interposed bentonite panel waterproofing are shown in Figure 4(c).

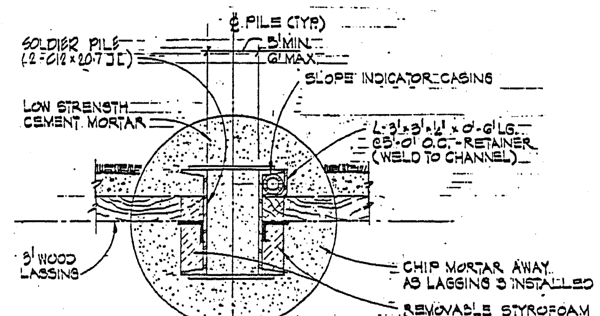

Fig. 4(a) Detail - Soldier Pile and Wood Lagging

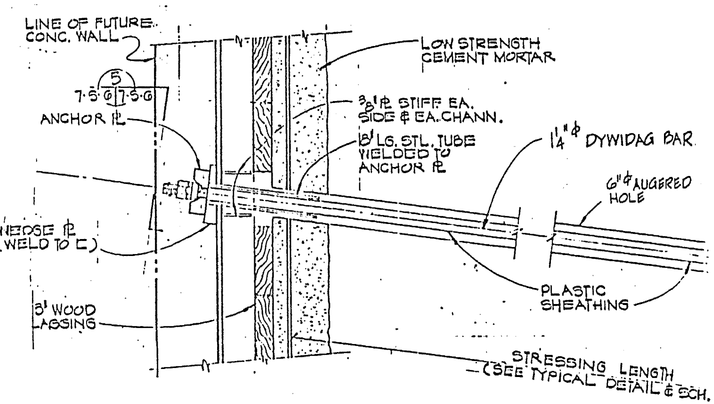

Fig. 4(b) Detail - Anchor Tie

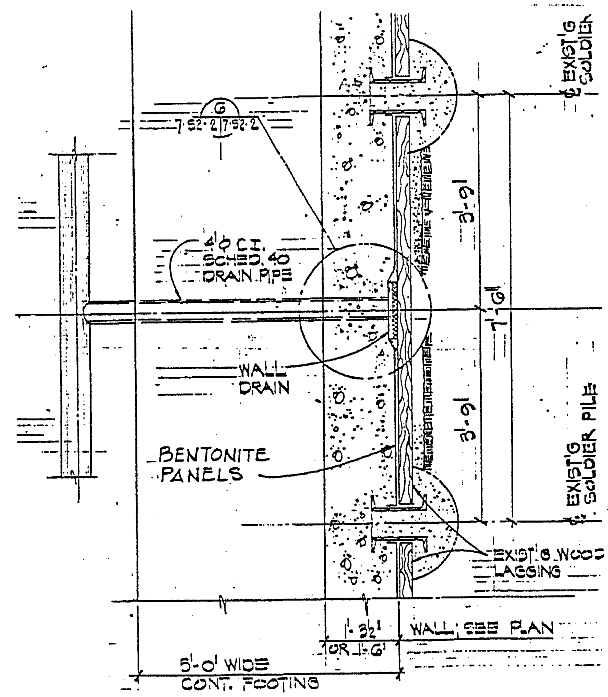

Fig. 4(c) Typical Detail - Finished Structural Wall

The tied anchor wall, 800 feet long, consisted of 119 steel soldier piles spaced at 7'-6" o.c., 3-inch wood lagging, 555 - 1-1/4 inch diameter Dywidag steel threadbar rods, and 15-1/2 to 18 inch reinforced concrete wall. The steel soldier piles, made up of two 12-inch channels assembled back to back, vary in height from 25 feet at the ends of the wall to 70 feet at the center. The number of tiebacks vary from two at the 25-foot minimum wall height to eight at the 70-foot maximum height. The tiebacks were spaced at 10'-0" to 5'-0" o.c. and staggered.

Design load on a tieback is 56 tons, except for the upper which included several at 36 tons. Each of the 555 tiebacks was load tested up to

115 percent of design load, except that 10 percent selected ties were tested to 134 percent of design load or 75 tons.

Load cells and slope indicators were installed at four selected soldier piles in order to monitor the behavior of the tied anchor wall during construction and approximately 180 days after completion (see Figures 3 and 4(a)).

Vertical wall drains between soldier piles were provided and Volclay Type C bentonite waterproof panels were fastened to the wood lagging prior to placing a reinforced concrete wall, 15-1/2 to 18 inches thick (see Figure 4(c)). The concrete wall was designed to span between soldier piles.

Figure 3 shows the location of four slope indicator tubes incorporated with the designated soldier piles. A total of 27 load cells were installed at these soldier piles, attached to the ends of the tie anchors as these were drilled and tensioned in the course of excavation. All were monitored at regular intervals during and subsequent to excavation and shoring. The remainder of this paper focuses on the findings and the response and/or conclusions in light of those findings.

LOAD CELL AND SLOPE INDICATOR OBSERVATIONS - DURING AND SUBSEQUENT TO EXCAVATION

The general shoring sequence involved installing all the soldier piles into 30-inch diameter, predrilled holes and backfilling with cement stabilized sand. Excavation proceeded in 5- to 10-foot increments, extending about 2 feet below the designated elevation of anchor ties in each increment. The ties were installed by drilling and grouting via a 6-inch diameter hollow stem auger and were tensioned generally three to four days thereafter. Subsequent to completion of each excavation increment but prior to tie tensioning, all the slope indicators and installed load cells were monitored. During the initial stages of excavation, slope indicators and load cells were also monitored <u>after</u> tie tensioning, prior to proceeding with additional excavation, but it became evident that tensioning the lower tier had insignificant influence on the loads on the previously installed ties at higher elevation.

The general interrelation between depth of excavation and changes in slope indicator profile and associated load cell readings are illustrated in Figure 5, at soldier piles SP-60. The following is particularly noteworthy:

1) The maximum lateral deflection was about 24 millimeters (i.e., 1 inch) and occurred in the course of excavating in Zone C, saprolite.

2) Discrete increments of lateral movements developed while excavating successive depth increments Zone C.

3) Lateral movements caused a significant increase in the tension forces in the adjacent tie anchor(s). Note in Figure 5, load on tie anchor at Elevation 240 increased from 57.2 tons immediately after installation, up to 62.9 tons when excavation reached nominal Elevation 230 and to 67.0 tons with excavation at 220. Also, tie load at Elevation 230 increased from 58.2 tons to 65.1 tons with excavation at 220. Similar response was noted at all the instrumented soldier piles in the course of excavating in Zone C (saprolite).

These movements had not been originally anticipated and corresponding tie load increases were of a magnitude to require corrective action. A review of the slope indicator profile did not suggest that the lateral movements were deep seated nor extending beyond the tied anchors. Also, there was no indication of moisture movement to cause swelling of the exposed face of the freshly excavated soil. On the other hand, the freshly exposed saprolite was steeply jointed and comprised of relatively hard chunks of weathered rock, with the fractured surfaces only weakly adhering to each other. One could observe during the smoothing of the exposed face preparatory to placing the wood lagging that soil readily broke off along the steep joints, particularly in Zone C. These circumstances were discussed with Professor George Sowers of Georgia Tech University and he advised of similar response he had observed when excavating in deeply weathered rock and residual soils in the Atlanta area and Piedmont in general. He speculated that deflections were due to mechanical rearrangement of the fractured materials and would stabilize after a relatively short period. He concurred with proceeding with a tie detensioning program and on-going tie load monitoring.

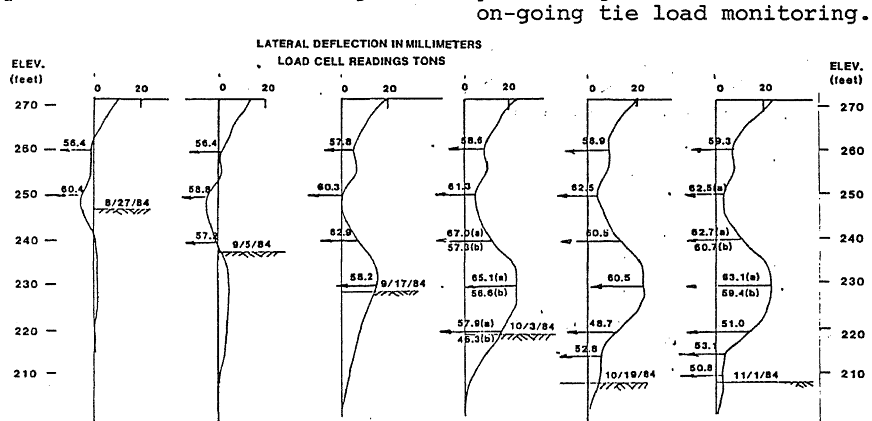

Fig. 5 Tie Loads - Lateral Deflection at SP-60

(a) cell load before detension
(b) cell load after detension

The scheme was as follows:

a) Commencing from a load cell indicating greater than 63 tons, the tensioning jack was reset at an adjacent tie and tension loaded until unseating the anchor nut, noting the force required. If force exceeded 63 tons, tie was detensioned to design load (i.e., 56 tons).

b) If a specific tie anchor was unseated by load <u>greater than 63 tons</u>, <u>all</u> adjoining ties were unseated and detensioned as per a) above.

c) If specific tie anchor was unseated by load <u>less than 63 tons</u>, further detensioning ties at that tier elevation were discontinued moving outward from the starting point in this test sequence.

Figure 6 indicates the location and number of times that specific tie anchors were unseated and detensioned in accordance with the above protocol. Of a total of 555 ties installed, 185 were detensioned either before or shortly after completion of excavation. It is clearly evident that the detensioned ties were concentrated in the Zone C saprolite strata as interpolated from the original soil investigation, Figure 3. Further, only 32 ties were detentioned twice, and these are clustered in the thickest portion of Zone C, between SP-40 and 75.

Although the reported initial unseating loads exceeded 75 tons in three instances, these are questionable since the jacking assembly registered unseating loads up to 6 tons greater than for corresponding load cell readings. The specific unseating and detensioning scheme was aimed at assuring a safe structure and jacking loads were too crude to define trends in load changes accurately subsequent to completion of excavation. However, Figure 7 indicates load cell readings for critical ties over about a 6-month period subsequent to completion of excavation and initial detensioning. The following is noteworthy:

a) The logarithmic time plot indicates a decelerating increase in tie loads over the observation period.

b) The load increase observed over 6 months ranges from 2,5 to 4.5 tons.

c) There is evidence that tie loads had reached equilibrium after 6 months.

d) Load cell readings were appreciably affected by environmental factors, particularly during the winter and spring observation period with indicated loads fluctuating up to 1 ton from the trend lines.

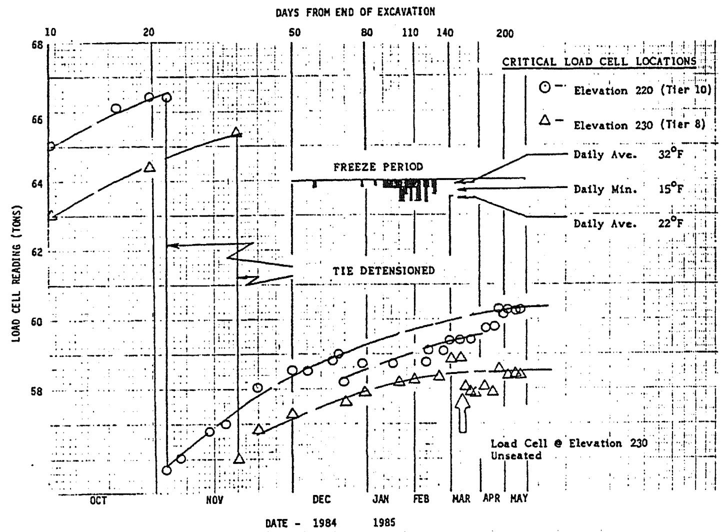

Fig. 7 Load Cell Readings After Excavation Completed - SP-82

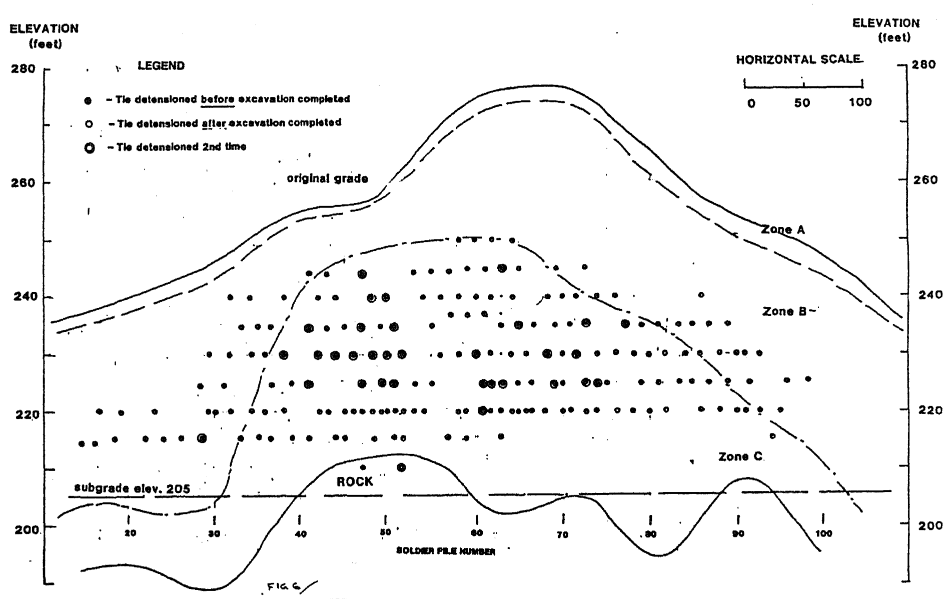

Fig. 6 Summary of Tied Anchor Detensioning

The observations as a whole indicate that lateral expansion in the course of excavation in Zone C can reach 1 inch and cause the tie loads to increase about 10 to 15 tons from installed values (i.e., from 56 tons to over 70 tons). Of that increase, from 2.5 to 4.5 tons was attributable to long-term expansion of Zone C, essentially completed in about 6 months at this site.

ANALYSIS OF EFFECT OF BENTONITE PANELS IN PERMANENT WALL

As indicated in Figure 4(c), the finished structural wall incorporated the temporary shoring, with bentonite waterproofing panels sandwiched between the wood lagging and the poured concrete wall. There had been a general concern that should the bentonite panel swell subsequent to the hardening of the concrete in the wall, tie loads in the affected areas might increase. Selected load cells were prepared to permit monitoring during the initial wall pours. Concurrently, load-swelling information was obtained from the panel supplier and load-compression relationship of the granular backfill behind the wood lagging determined.

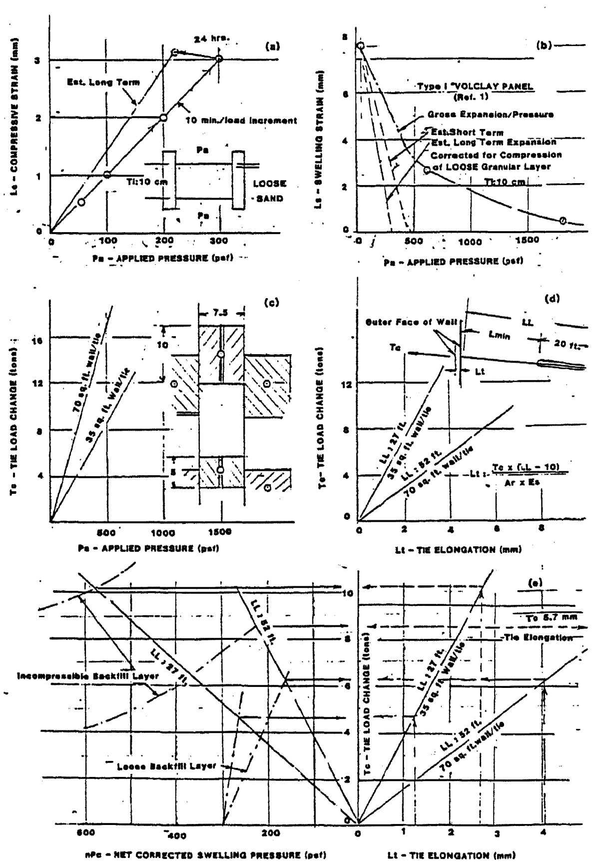

Fig. 8 Effect of Bentonite Panel Expansion On Tie Anchor Loads

Figure 8 indicates the essential factors and the corresponding parameters used to estimate the tie load increases attributable to swelling of the bentonite panels. Figure 8(a) shows relation between compressive strain and applied pressure for loose granular soil deemed representative of backfill as placed. Figure 8(b) plots the ultimate swelling strain versus applied confining pressure for the bentonite panel. As indicated in Figure 4(c) the strain corresponds to inward movement of the concrete wall relative to the inner surface of the wood lagging. This relation is corrected as indicated for the estimated short- and long-term outward strain of the wood lagging as it compresses the backfill at the same and corresponding applied pressure. Figures 8(c) and (d) show interrelation between changes in net swelling pressures and corresponding changes in tied anchor loads and strains for the applicable wall areas of influence. Finally, Figure 8(e) interrelates the tie load changes and rod length changes and associated swelling pressures and attendant wall movements, corrected for assumed extremes of compressibility of backfill. It is noted that for the range of effective lengths of tie rods (i.e., 27 to 52 feet) and associated areas of wall supported (i.e., 70 to 35 square feet, respectively), the calculated tie load increases ranged from 4.5 to 6 tons up to an outer limit of 8.5 to 10 tons, depending on the compressibility of the backfill retained by the wood lagging.

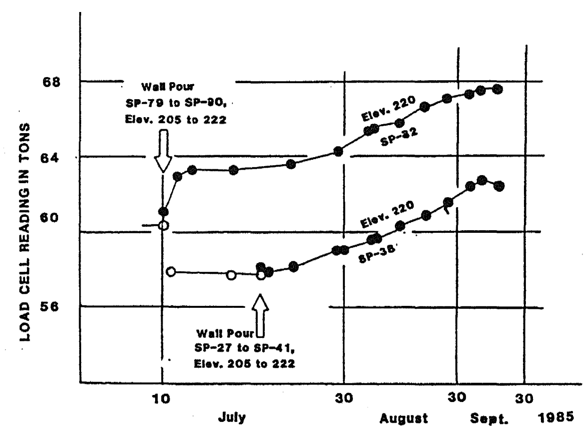

Fig. 9 Tie Anchor Loads After Wall Pour

These estimates appear to bracket load cell readings summarized in Figure 9. We note the loads increased from 4 to 5 tons over a 2.5 month period, tracing an "S" shaped curve over a log time scale during which it is presumed that the bentonite was absorbing moisture from the adjacent poured concrete. It also appears that swelling was completed in about 2 months in these circumstances. To the writers' knowledge there has been no visual manifestation of additional swelling or tie load increase during the succeeding two years.

SUMMARY

Observations of a tied anchor retaining wall, ranging up to a 70-foot height in variable residual soil and weathered rock indicated unanticipated variable lateral movements up to 1 inch due to a rapid and limited expansion of the partially weathered rock (i.e., saprolite) during excavation. This was evidently due to localized mechanical rearrangement as the heavily jointed and fractured stratum was first exposed. Attendant anchor loads increased from a design load of 56 tons to over 70 tons during and immediately subsequent to excavation. Affected tie anchors were subsequently unseated and partially detensioned one or more times where necessary to re-establish design load values. Tie loads were observed to increase at a sharply reduced rate over about 6 months after completion of excavation, limited to 4.5 tons above the original 56 tons.

Analysis indicated that bentonite waterproofing panels incorporated behind the structural wall could swell sufficiently to appreciably increase the affected tie anchors. Load cell readings increased over a 2-month period, leveling out at a maximum 4-ton increase. Depending on the compressibility of fill behind the wood lagging, analysis suggests increases might reach 8 tons in particular circumstances.

ACKNOWLEDGEMENTS

The original geotechnical investigation and geological analysis for the project was performed by Dames and Moore, John Kittridge, Project Manager. Woodward Clyde Associates supervised installation and monitored the load cells and slope indicators.