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Construction and performance of a permanent earth anchor (tieback) system for the Stanford Linear Collider*

MARK N. OBERGFELL, P.E., A.M. ASCE, Chief Civil Engineer
Stanford Linear Accelerator Center, Stanford University, Stanford, California 94305

ABSTRACT: The Stanford Linear Collider is the newest addition to the high-energy physics research complex at the Stanford Linear Accelerator Center. One of the many unique features of this project is the large, underground pit, where massive particle detectors will study the collision of subatomic particles. The large, open pit utilizes nearly 600 permanent earth anchors (tiebacks) for the support of the 56 ft (17 m) high walls, and is one of the largest applications of tiebacks for permanent support of a structure. This paper examines the use of tiebacks on this project with emphasis on their installation and performance.

1 INTRODUCTION

The Stanford Linear Collider (SLC) is a new, high-energy physics project located at the Stanford Linear Accelerator Center (SLAC), about thirty miles south of San Francisco on the peninsula between San Francisco Bay and the Pacific Ocean. SLAC is a high-energy physics research center operated by Stanford University for the United States Department of Energy (DOE). SLAC is dedicated to basic, fundamental, particle physics research. The main component of SLAC is the two-mile (3.2 km) long linear accelerator [housed in a concrete tunnel 25 ft (7.6 m) underground], which creates high energy electron and positron beams. The Collider will take the two beams from the accelerator, bend them around two tunnel arcs and collide the beams head on in the experimental hall pit. The two-mile accelerator, along with the pit excavation (at bottom of photo), is shown in Figure 1. In the pit massive, 3300 ton detectors will study the subatomic particles resulting from the collision.

Figure 1. Aerial Photograph of SLAC site

In general, tiebacks have gained widespread acceptance for use in temporary earth-retaining structures. They eliminate the need for large, open excavations with sloped sides, or the need for internally braced sheeting. For large, deep excavations, tiebacks become economically feasible. Relative to conventional cantilever retaining walls, tieback walls do not need large footings and the costly over-excavation and backfill. The use of tiebacks has become popular for tight construction sites in urban areas where space is limited by adjacent structures, and in subway, bridge abutment, and highway retaining wall construction. Their use in temporary structures, and more recently in permanent structures, has been documented by Weatherby (1982) and Anderson (1984).

2 DESIGN

2.1 Preliminary Studies

During preliminary feasibility studies for this project, several schemes for the pit were considered. The pit itself is 233 ft (71 m) long, 65 ft (20 m) wide, and 56 ft (17 m) deep. A pit of this depth with the requirements for clear floor space presented a few problems in the

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design of the high retaining walls. The depth of the pit was dictated by primarily two factors: 1) the tunnel had to be at least 40 ft (12 m) below ground at an adjoining park, and 2) the tunnel could have a slope of no more than 10%. Therefore, the pit had to be deep underground to intercept the two tunnel arcs. Conventional cantilever wall construction would have required walls 4 to 6 ft thick (1.2 to 1.8 m) in addition to the over excavation and temporary retaining walls with soldier piles and tiebacks. At that point, it was a logical move to consider permanent tiebacks for this project.

When the tieback system was first proposed, it was met with a fair amount of skepticism and heavy questioning. This was due in part to the relative newness of the technique and limited experience in the United States for permanent wall construction as an integral part of a building. It should be noted that a fair amount of questioning came from the physics and research community and not from the engineering and construction community. Permanency of the tiebacks and their ability to hold the loads without excessive movement were the primary topics of discussion. After numerous meetings, contacts with industry representatives and with consultants in Europe familiar with tiebacks, the system was finally approved for design and construction. In addition to conservative engineering design practices, several other items were implemented to assure the adequacy of the system. The anchors were to be double-corrosion protected, and an extensive testing and monitoring program was to be undertaken. These measures are more typical of tiebacks in cohesive soils, rather than the competent sandstone at this site. Other measures will be discussed later.

2.2 Details of Project

To my knowledge, this is the largest permanent building tieback installation in North America. The pit, as shown in Figure 2 at completion of construction, consists of the central area, intercepted by the two tunnel arcs, where the particle collisions will be observed, and the east and west garage areas where the huge particle detectors will be assembled and serviced. In the central pit area, the large struts between the two walls were an extra measure of protection to insure that the walls would not move inward and pinch the removable concrete shielding walls that would be installed later. Around the pit, there are large rim beams designed to support the weight of the heavy detector parts as they are moved into place and lowered into the pit, and to support the 39 inches (1 m) thick concrete shielding planks. To avoid placing a surcharge on the walls, the rim beams, as well as the main structural steel for the high bay area, are supported on belled caissons which extend to a depth below the pit slab.

Figure 2. Pit at completion of construction

There are a total of 584 tiebacks on the four pit walls. They are installed on soldier piles spaced at 5 to 8 ft (1.5 to 2.4 m) on center, with a total of seven tiebacks on each soldier pile. The soldier piles consist of double steel channels embedded in concrete. The walls are conventionally reinforced, cast-in-place concrete, 15 inches (38 cm) thick. A cross-section through the pit (showing the tunnels, tiebacks, and high bay steel structure above) is shown in Figure 3.

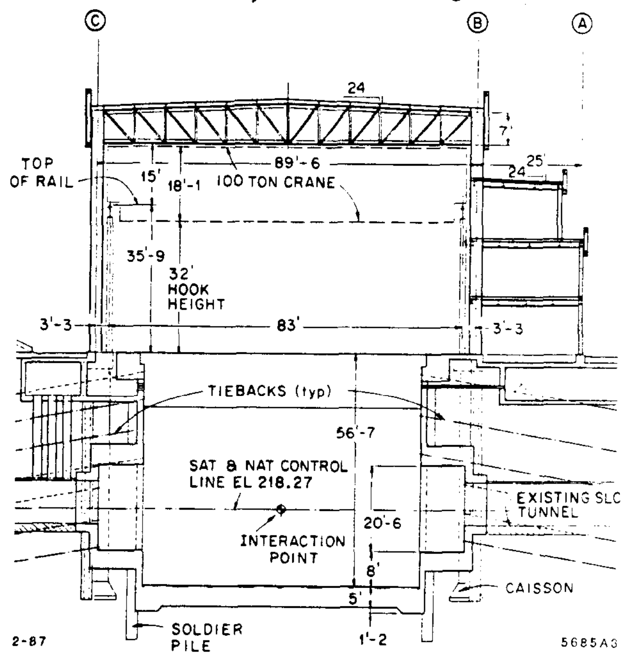

Figure 3. Cross–Section of Pit

The anchors are double-corrosion protected, 1–3/8 inches (3.5 cm) in diameter made of ASTM A722 steel. The corrosion protection system consists of grout encased in a corrugated PVC sheath. This entire assembly was installed as a unit in the drilled hole, then grouted in place. The anchor length of the tiebacks is 25 ft (7.6 m). The predominant soil

Table 1. Engineering properties and geotechnical design parameters for miocene sandstone at the site.

Design parameter	Sandstone type: Uncemented to weakly-cemented	Sandstone type: Moderate to well-cemented
Effective cohesion C'	0 psi	16 psi
Effective friction angle ϕ'	35°	42°
Dynamic effective friction angle ϕ'_E	38°	46°
Density	125 pcf (19.6 kN/m³)	130 pcf (20.4 kN/m³)
Coefficient of active earth pressure K_A	.27	.20
Dynamic coefficient of active earth pressure K_{AE}	.24	.16
Coefficient of lateral earth pressure at rest K_O	.43	.33
Compressional wave velocity at low strain level V_P	1928 fps (588 m/s) (top 35 ft) (10.6 m)	3370 fps (1027 m/s) (35 ft–90 ft) (10.6 m–27.4 m)
Shear wave velocity at low strain level V_S	1225 fps (373 m/s) (top 35 ft) (10.6 m)	1952 fps (595 m/s) (35 ft–90 ft) (10.6 m–27.4 m)

at the site is weakly to well cemented miocene sandstone; as such, it was an ideal candidate for permanent tiebacks.

2.3 Testing Program

In order to determine the suitability of tiebacks in the intended location and to determine design parameters, a field testing program was undertaken. A total of five tiebacks were installed at the project site in the initial open cut excavation. The first prototype failed miserably, and this once again led to questioning of the entire system. Upon closer examination, it was determined that the drilling procedure was to blame; water was used for the drilling and this caused a thin layer of mud to form along the hole. Subsequent prototypes gave the desired results after the drilling procedure was modified. Two of the tests, using the final configuration of the tieback, yielded a load of 189 kips (840 kN) before failure. This correlated to a bond stress of approximately 33 psi (228 kPa). After the load testing, two of the tiebacks were locked off at the design load and monitored for creep for a period of four months. When projected out, this showed a loss of less than 10% in the initial load over a period of 25 years.

2.4 Design Parameters

In addition to the precautions taken with the tiebacks themselves, design loads were conservatively determined, and some of the beneficial effects of cemented sands were neglected. The walls and anchors were designed for static and seismic earth pressures. The static pressures are active pressures using a combined rectangular and triangular pressure distribution. The walls were designed for a major earthquake resulting from the San Andreas Fault, which is located approximately 3 miles (4.8 km) to the west of the site. The design earthquake is expected to produce horizontal ground accelerations of six-tenths gravity (0.6 g). The soil pressure diagrams and the design parameters are shown in Figure 4 and Table 1 respectively. The basis for the design criteria was the active failure wedge (treating the material as a soil), which was also used in part to determine the unbonded lengths of the anchors. The earthquake pressures were developed using a pseudostatic force developed from the active wedge, which was then uniformly distributed over the wall (Tudor 1984a). The combined static and seismic loads resulted in a total working design load of 120 kips (534 kN) for a typical tieback.

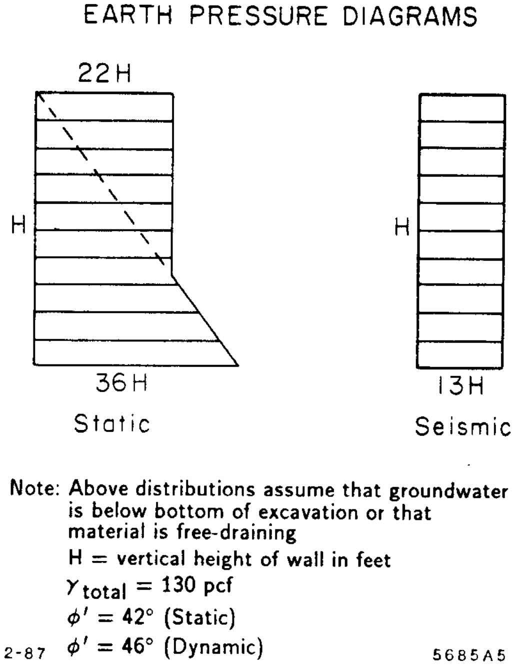

Figure 4. Earth Pressure Diagrams

The anchor capacity was estimated using the following equation:

$$P = \pi DLQ$$

where

P = allowable anchor load ,

D = anchor diameter ,

L = anchor (bond) length ,

Q = bond stress .

The bond stress was conservatively estimated at 5000 psf (239 kPa) for soft, sedimentary rock.

In addition to the above criteria, an overall stability analysis was performed assuming that the tiebacks tie the soil together into a rigid mass. In other words, they prestress the soil so that the failure wedge is prevented from developing Appropriate factors of safety were considered for tieback stress, failure of the active wedge, as well as creep of the anchors. The design also allowed for any one anchor to fail, except the top anchor, without causing overall failure. The walls themselves were designed using working stress, rather than ultimate strength procedures for concrete.

3 CONSTRUCTION

Construction of the two tunnel arcs began in the Fall of 1983, using two road-header tunneling machines purchased from Germany for use on the project. A small portion of the work was done by conventional cut and cover methods in areas where the tunnel was at a shallow depth. At the same time as the tunnel work, a contract was let to begin the initial open cut excavation at the site of the Collider Hall and pit. This involved removing the large amount of overburden down to the future grade level of the building. The actual work for the building and pit began in the Fall of 1984 as the piles and caissons were drilled and placed, and the excavation for the pit started. At shallow depths, excavation was accomplished using a hydraulic excavator with the soil removed via dump trucks and a ramp out of the pit. At greater depths, the soil was removed with a clamshell bucket. Wire mesh was installed between soldier piles to prevent sloughing and to catch loose debris. No wood lagging was used.

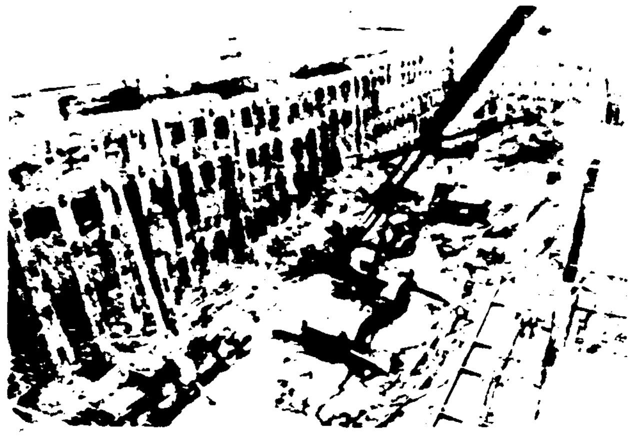

Figure 5. Pit Excavation

The tiebacks were installed in layers as the excavation proceeded. The general procedure was to excavate, install tiebacks and grout, and then proceed down to excavation of the next level when the anchor grout had gained sufficient strength. The holes were drilled using an 8 inch (20 cm) diameter, hollow-stem auger mounted on a tracked rig, with the auger modified to scarify the hole. The excavation and the drilling rig are shown in Figure 5. The tunnel has just been uncovered in the center of the photo, while the tiebacks and soldier piles are visible around the perimeter. Note the drilling rig and the stratification of the sandstone. With this type of set-up used by the contractor, including the use of no drilling water, the anchors developed bond stresses nearly twice that determined from the testing program. A pull-out test done by the contractor in the better sandstone at the pit yielded a bond stress of 65 psi (448 kPa). Once the hole was drilled, the tiebacks were installed in the hole and grouted. Next, the tiebacks were proof-tested to 133% of the design load, and then locked off. A small percentage of the tiebacks were also subjected to performance tests. Concrete wall construction began once the excavation was completed and the base slab was poured.

4 PERFORMANCE AND MONITORING

4.1 System Description

An extensive monitoring program is underway to measure the performance of the tiebacks. Load cells have been installed on 67 tiebacks at various locations on the four walls, and are currently being monitored. During the early monitoring stages, ongoing construction activities limited access to the pit. It was difficult to get access to certain areas due to the large amount of work going on in such cramped quarters. Once construction was complete, the onslaught of the equipment installation further hampered monitoring. It took many reminders to prevent the installation crews from permanently blocking access to the tiebacks with pipes, conduit, and cable trays. Once construction and equipment installation was completed, monitoring could be carried out on a regular basis.

As a part of the construction contract, the contractor was required to perform lift-off tests and any necessary re-tensioning after completion of the excavation, immediately before pouring the concrete walls, and just before completion of the contract (Tudor 1984b). These measures were employed to address the anticipated creep and the adjustment of the soil due to excavation and imposed loads. In addition to the load cells, extensometers were placed on each wall to act as benchmarks for detecting and mapping movements of the walls. These extensometers consist of steel rods anchored into the sandstone 10 ft (3 m) beyond the end of the tiebacks.

4.2 Construction Monitoring

Shortly after the excavation intercepted and passed the previously bored tunnel, cracks began to appear in the fiber-reinforced, shotcrete tunnel lining. There were cracks in both the north and south tunnels, but the ones in the south side were larger and more numerous. Extensometers were installed in the south tunnel to monitor the movement of the cracks. Maximum recorded movements were 0.18 inches (4.6 mm), with the average approximately 0.08 inches (2 mm). Movement stabilized by the time excavation was completed, but it did cause some concern. The cracking and movement was undoubtedly due to the dip of the geological bedding in towards the pit at the south side, and the fact that the tiebacks directly above the tunnel had not been installed or stressed until the excavation was below the level of the tunnel. This was necessitated because the tiebacks had to be installed almost horizontally so they would not intercept the tunnel. The contractors rig had to be at a lower level in order to drill nearly horizontal (the typical slope of a tieback is 20° from the horizontal).

Lift-off tests and load cell monitoring during construction found that loads had increased on the order of 5% since initial installation and lock-off. The largest increases were seen in the east wall and in the easterly half of the south wall. An independent geotechnical consultant's review attributed the increase to the surcharge affect of the 2:1 slope rising to the east and south of the site. The slope rises on an average of 50 ft (15 m) in elevation above the top of the pit, and is setback approximately 30 ft (9 m) from the edge of the pit. The north and west sides of the site are fairly level and little increase was seen in tieback loads in these areas except for the northwest corner of the pit. Contrary to earlier expectations of creep problems, there appeared to be none.

Although there was some concern, it was decided to wait and continue to monitor the load cells. The unfavorable surcharge, combined with the relaxation and readjustment of the earth due to the excavation and overburden removal, were the prime suspects. It was the general consensus that as activities subsided and equilibrium was reached, the load increases would stabilize and the tiebacks would perform as designed.

4.3 Post-Construction Performance

After construction was completed and the facility was turned over to SLAC, a reading was made on all load cells once again. The readings showed a continuing increase in tieback stresses. Some tiebacks were approaching the manufacturer's recommended maximum lock-off load of 165 kips (734 kN). Figures 6–8 show typical load-time histories for the tiebacks. The tiebacks were initially installed and locked-off at 140 kips (623 kN) to allow for some creep and resultant decrease in anchor tension on the assumption that the stress would decrease to the design of 120 kips. Again, the largest increases were seen in the south wall. There was immediate concern, and after several meetings with the consulting engineers and geotechnical engineer, it was decided to embark on a program of reducing all tiebacks on the south, east, and west walls to $\approx$ 125 kips (556 kN). Note from Figure 7 that the east wall could not be reset at the same time due to the large detector blocking access. The north wall however was not seeing such a rapid rate of increase. Loads had held fairly steady at the 140 kip level, and it was decided to leave the north wall alone. Creep was definitely not a problem and the factor of safety in the original design was being encroached upon by the ever increasing loads. Although construction activities had subsided, the tiebacks and earth were still adjusting to the new conditions. Installation activities, such as the rolling-in of the large detector and other heavy equipment use around the pit probably contributed to the changes still being experienced.

Since the program of resetting the tiebacks in the summer of 1986, the load increases have levelled out. Installation activities have subsided as the facility is about ready to begin operation. Readings taken in January, 1987, have shown no significant increases. In the south wall, loads have held steady at or near the 125 kip (556 kN) lock-off load. The north wall has experienced no change whatsoever; readings are identical

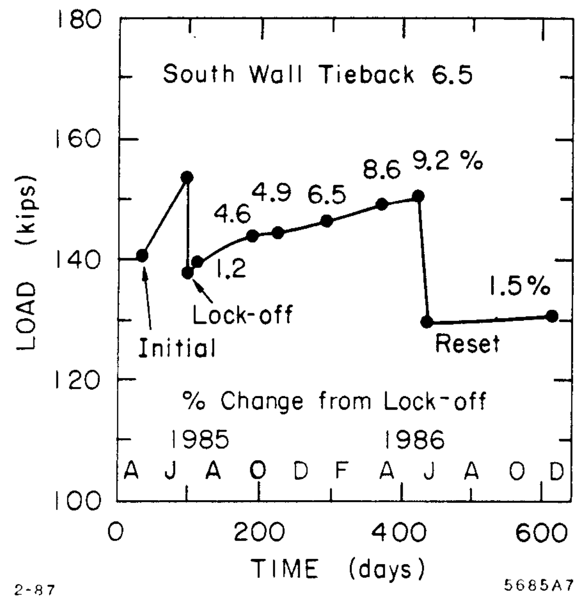

Figure 6. Typical South Wall Tieback

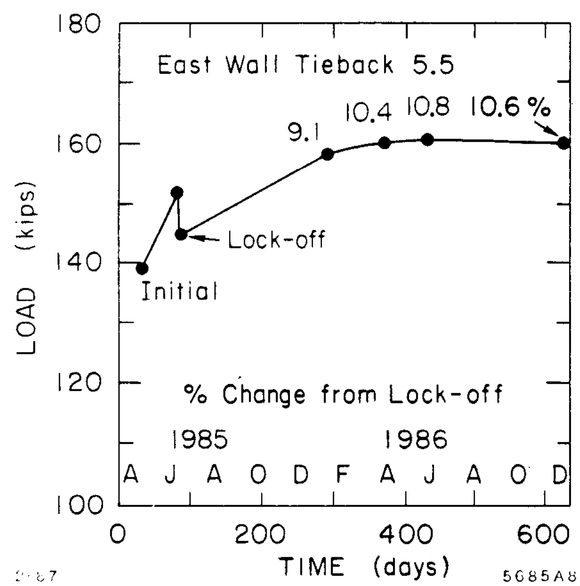

Figure 7. Typical East Wall Tieback

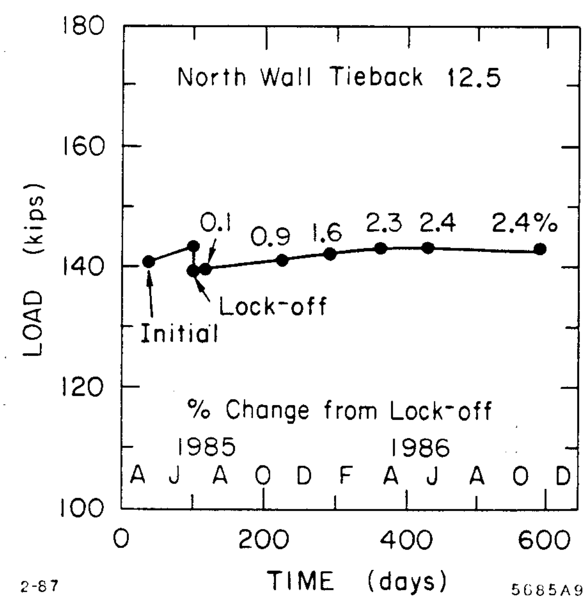

Figure 8. Typical North Wall Tieback

to those taken in the summer of 1986. The east and west walls have stabilized also. To this date, the extensometers in the walls have not been used to map any movements. There have been no indications of movement in the walls and there have been absolutely no cracks or other signs of distress. Implementation of a surveying system to map the relative position of the wall and to detect movements on the order of 1/16 inch (1.6 mm) would be expensive, not to mention the clutter of piping and other equipment that make lines of sight difficult. Since the loads have stabilized, confidence in the system of tiebacks for permanent use has been restored. There have been no moderate or major earthquakes to give the system a true design test, although this will likely happen during the life of the project. Monitoring will be continued on a regular basis; eventually there will be a complete and thorough record of the performance of this unique engineering and construction accomplishment.

5 CONCLUSIONS

- The initial selling of the tieback system for permanent construction to a non-engineering (but highly scientific and educated) community was a difficult task due to the limited amount of information on the system for permanent use.
- Corrosion concerns and long-term performance characteristics were the primary questions that had to be addressed in the initial design.
- Marked differences were seen between field prototypes and the actual installed anchors. Higher bond stresses were attained by the contractor, and creep proved to be no problem.
- The removal of the overburden, the sloping surcharge, and the dip of the geological bedding played significant roles in the changes seen in the tieback stresses. Equilibrium was not reached as soon as expected. Load increases were not expected.
- Recent performance has been favorable. Loads have stabilized and the system is performing as intended.
- The use of tiebacks resulted in a substantial cost savings, estimated to be approximately $1,600,000 (U.S.) by Mueller (1984).
- SLAC is a pioneer in the physics and research field with many important discoveries. It was appropriate, therefore, that engineering and construction should play an important role in making this new project possible. It is hoped that the experience and data collected on the tieback system will be helpful to others on future projects.

ACKNOWLEDGEMENTS:

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- Geotechnical Engineer: Earth Sciences Associates, Palo Alto, California
- General Contractor: Dickman-Nourse, Inc., Mountain View, California
- Tieback Subcontractor: DBM Contractors, Inc., Federal Way, Washington

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