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PERMANENT SOIL NAIL EARTH RETENTION SYSTEM, STANLEY HALL REPLACEMENT PROJECT, UC BERKELEY

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Abstract

The new Stanley Hall building will house the center for bio-engineering and technology research for the UC Berkeley campus. The structure is located at the northeast part of campus, adjacent to Donner Lab, on the west side of Galey Road, approximately 600 ft. from the Hayward Fault. The building site has a complex geological formation. To the east there are clayey soil shale formations, and to the west there is greywake shattered sandstone. The water table is at 10 ft. below grade.

The structure will consist of 6 levels below grade and 8 levels above grade, which calls for an 83 ft. deep excavation on the northeast side. There is an approximately 35 ft. grade differential between the east and west sides of the building. Because of this severe grade differential, as well as high water tables, the lateral forces on the building due to uneven earth pressures would have been 2 times the magnitude of the seismic forces.

To mitigate the lateral earth pressures, a permanent Soil Nailed Wall, integrated with the structural system of the building was designed. PB&A, Inc.'s in-house design program "Winslope" was utilized to design the system and the data obtained from a rigorous monitoring program, were used to calibrate several computer runs using Plaxis program.

STANLEY HALL: OLD AND NEW

Named in honor of Professor Wendell Meredith Stanley, Chair of the Biochemistry Department and Director of the Virology Laboratory in 1952, Stanley Hall housed University of California, Berkeley faculty offices in the Department of Molecular and Cell Biology. Over 50 years old and no longer conforming to seismic codes, the old building was demolished in 2003 to make way for a state-of-the-art Biosciences and Bioengineering facility.

At 270,000 sq. ft., the structure, designed by the architecture firm, Zimmer Gunsul Frasca Partnership, will be more than 4 times the size of its predecessor. It will serve as an interdisciplinary science hub, providing cutting edge technology and equipment, and flexible modular laboratories for the research and study of chemistry, microbiology, and nanotechnology.

The building will also house the newly established Department of Bioengineering, as well as research strains of the departments of Chemistry and Physics. Planned for completion in January, 2006, the $150 million structure will serve as the new standard for the future of science research and instructional facilities.

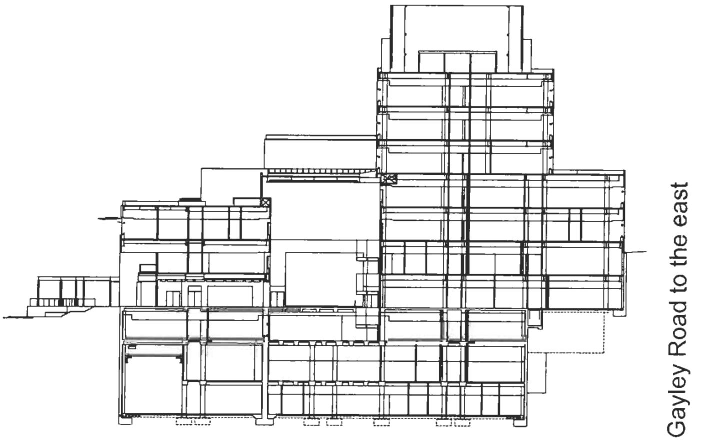


Figure 1: Building Cross-Section

Working With a Complex Site

Located at the northeast corner of the UC Berkeley campus, the structure is built into site which slopes up to the northeast, with elevations rising from 350 ft. at Hearst Mining Circle to the west, to 390 ft. at Gayley Road to the east. The structure includes several setbacks as it climbs the elevation, as shown in Figure 2. The building perimeter has varying degrees of landscaping, including a variety of bushes and trees.

The building itself includes seven levels of steel frame construction above grade, three levels of concrete frame construction below grade, and incorporates a large skylight atrium on the entry axis.

Between the east wall, which was excavated down to 83 ft. deep (see Figure 2), and the west wall, there exists a grade differential of almost 40 ft., causing uneven earth pressure to exert extreme lateral forces on the building, up to 2 times the magnitude of the seismic forces. This severe grade differential, combined with a complex soils make-up, the geometry of the site, high water table, and the presence of potential seismic shifting required an innovative Earth Retention System design to ease forces on the structure.

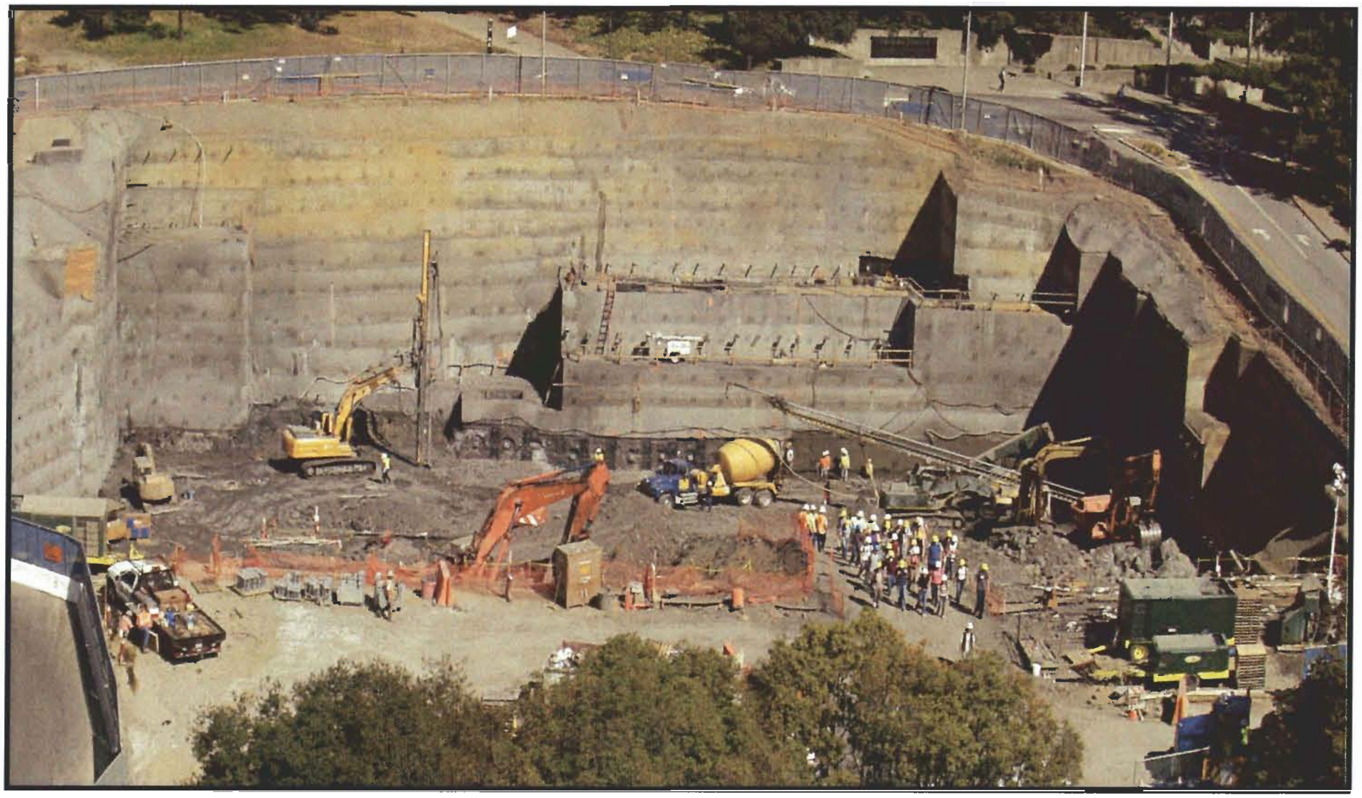


Figure 2: Stanley Hall Excavation, east wall view

To accomplish this, the steel-framed superstructure is isolated from the sloping hillside along the east wall, and portions of the north and south sides, by the use of a permanent soil-nailed earth retention system. The permanent system, designed by PB&A, Inc., is integrated into the structural system of the building below the lowest grade, and serves to mitigate these lateral earth pressures.

GEOTECHNICAL CONSIDERATIONS

The Berkeley Hills lie on the Hayward fault, which signifies the boundary between the San Francisco Bay block and the highlands of the East Bay Hills. At this point it is observed that an abrupt change in topography occurs as the steep hills flatten to the west and into the San Francisco Bay basin. Much of the Berkeley Hills are covered with native colluvium, as well as man-made fills. This is predominately underlain by rocks comprised of sandstone, shale, and conglomerate, the most common of which is Franciscan bedrock, including graywacke sandstone and shale.

According to the geotechnical investigation performed by project geotechnical engineers, MACTEC, the site lies within the Coast Range geomorphic province. The area is dominated by northwest-trending faults, folds, and numerous other geologic structures. While the Hayward fault is the closest major active fault, a number of other local faults were considered able to induce significant shaking at the site. These faults include the Concord, Calaveras, San Andreas, Greenville, West Napa, San Gregorio, and Rodgers Creek.

The site's proximity to the main trace also required consideration of secondary or branch faults, specifically the Louderback Trace which brushes the site as close as 6 to 8 ft. at a North West to South East direction at the north east corner of the site.

This secondary fault, or shear zone, runs across the site, and according to a 1988 Harding Lawson Associates (HLA) report (completed as part of a fault investigation for the UCB Foothill Student Housing) forms a zone of thin, continuous shears that generally strike northwest and dip northeast. The report observed shears in both bedrock and 100,000-year-old colluvial soil exposed in trench excavations, but younger soils overlaying the shears were not offset by faulting. Based on this information, HLA concluded in its 1988 report that the Louderback trace is not seismically active.

The Stanley Hall geotechnical investigation observed a substantial change in rock character from the eastern borings to the western borings. Higher drilling rates were recorded at the borings on the west side of Stanley Hall, associated with harder graywacke, whereas the eastern borings encountered sheared shales interbedded with greywacke and some weaker sandstones, producing lower drilling rates. While upper fills in Borings B-3 through B-7 (see Fig. 7 for boring locations) include man-made superficial fills from 2 to 14 ft. thick, Borings B-1 and B-2 did not encounter any significant fills. It was concluded that this change in character was likely related to the Louderback trace.

Initially, for purposes of design of the Earth Retention System, it was assumed that this change in soil make-up essentially divided the site in half at a contact boundary, which ran north to south across the middle of the site. However, after excavation, it was discovered that the contact actually runs at a diagonal from near the northwest corner, across the site to the southeast, and that in fact rock hardness, strength, fracturing, as well as degree of weathering increases gradually from east to west across the entire site.

Adding to this variance in soil character, several borings intersected a paleo channel, the remnant of a buried stream channel that extends through the southern half of the site. Networks of these paleo channels, which are associated with Strawberry Creek, run down through the UC Berkeley campus. The channels have been filled over time with natural sediments, as well as man-made fill from campus construction projects. Right lateral slip movement along the Hayward fault has offset these channels, which is still evidenced by the present-day flow of Strawberry Creek as it runs out of Strawberry Canyon and then south of Hildebrand Hall.

The particular channel crossing the Stanley Hall site was mapped by Mr. Pat Williams of Rutherford & Chekene, Consulting Engineers on the 1998 Hearst Mining Building Project (which sits 150 feet northwest of the Stanley Hall site). In Boring 6, at the west edge of the existing Stanley Hall, drilling was slightly impeded due to the presence of boulders, believed to have been washed down the channel from slopes to the east.

Such a geologically complex site posed numerous challenges, not the least of which was determining the most suitable Earth Retention System to use for the project. It was determined in the geotechnical investigation that a system that could be employed as shoring during various stages of excavation and then be incorporated into the permanent structure of the building would be the most effective. Ultimately, Permanent Soil Nailing was chosen as the preferred solution.

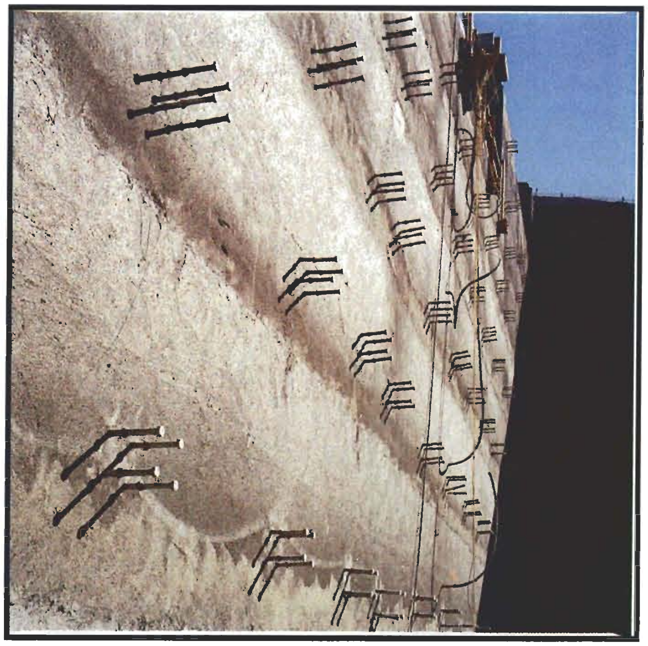


Fig. 3: Soil Nail Matrix

mass of soil, acting much the same as a gravity earth dam to resist the lateral pressures of the soil behind the boundaries of the Soil Nails. The passive reinforcements develop their strengthening action as the ground deforms during wall construction. (See Fig. 4 for an example of a typical Soil Nail Wall).

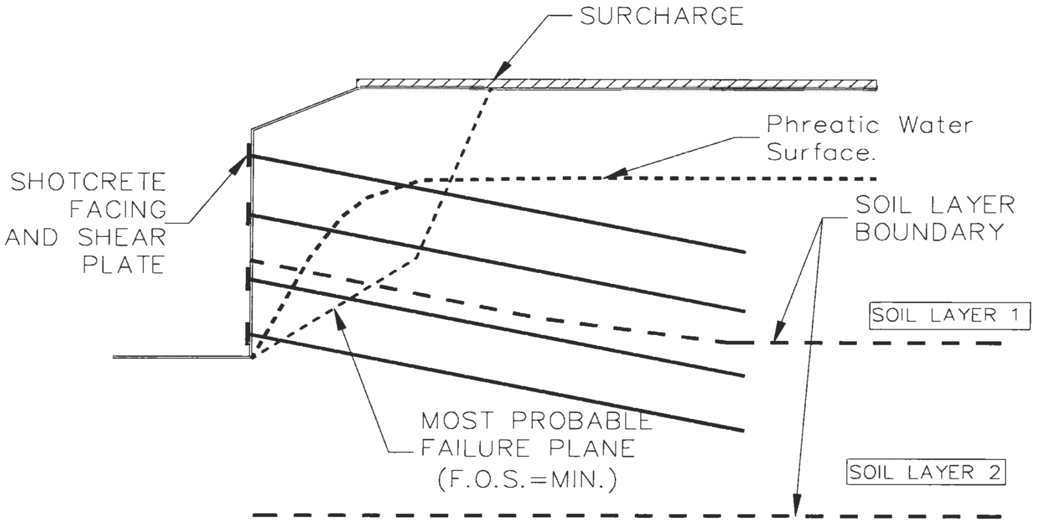


Fig. 4: Typical Soil Nail Wall Section

PERMANENT EARTH RETENTION SYSTEM

Mr. Tom Lauck, Principal Engineer with Rutherford & Chekene, whose long-standing relationship working with Mr. Pirooz Barar, S.E. and founder of PB&A, Inc. extends back to their days working with Skidmore Owings & Merrill, collaborated with Mr. Barar to develop a method of Permanent Soil Nailing earth retention, wherein the system could ultimately be integrated into the permanent structure of the building.

The basic concept of Soil Nailing, also referred to as "In Situ Reinforced Earth," is to strengthen a slope or excavation wall consisting of existing ground (Foundation Material) by means of installation of steel rods ("Soil Nails"), in grouted holes. The Nails are installed in the pattern of a matrix with a spacing not-to-exceed 6 ft. X 6 ft. The length of the Nails is typically between approximately 80% and 120% of the height of the excavation. When the Nails are installed, it creates a homogeneous and reinforced

Soil Nailing uses a "top-down" construction technique with one level excavated, a row of Nails installed, and a layer of protective material applied to the face of the excavation (typically 4" thick shotcrete reinforced by welded wire mesh).

If it is feasible for a given project, the benefits of Soil Nailing are many, especially if the project is a large complex structure, with limited accessibility, accelerated construction schedule, or cost limitations. Soil Nailing allows for elimination of high capacity structural facing, elimination of soldier piles or piers, ease of construction, reduced construction time, and greater system redundancy.

Decision to Use Permanent Soil Nailing

Stanley Hall proved to be a perfect candidate for the use of Permanent Soil Nailing, as the excavation site sits in the palm of a sloping hillside in a potentially very active seismic zone, and a severe grade differential at the site could exert extreme lateral loads from the unbalanced earth pressure. These loads amounted to about 4 million lbs., or roughly 2 times as much as the seismic load the building was designed to handle. These factors, coupled with the importance of waterproofing behind the wall, high water table, and the very complex geometry of the excavation (due primarily to architectural plans) rendered conventional earth retention systems untenable.

Earth Retention Systems, similar to any other structure have their own modes of vibration and dynamic characteristics. In the case of Stanley Hall, the dynamic characteristics of a 10 story building would be very different from that of an 83 ft. high Soil nailed wall that surrounds it. In other words, in the scheme that the Earth Retention System is separated from the main structure below grade, although the separation is only the thickness of the waterproofing, even in a moderate earthquake the differential strains of the two systems would not only damage the waterproofing but also would make this scheme structurally undesirable.

Considering the above facts, it was decided to employ a Permanent Soil Nail Earth Retention solution separate the wall from the superstructure and interconnect the soil nail wall with the basement wall below the lowest grade.

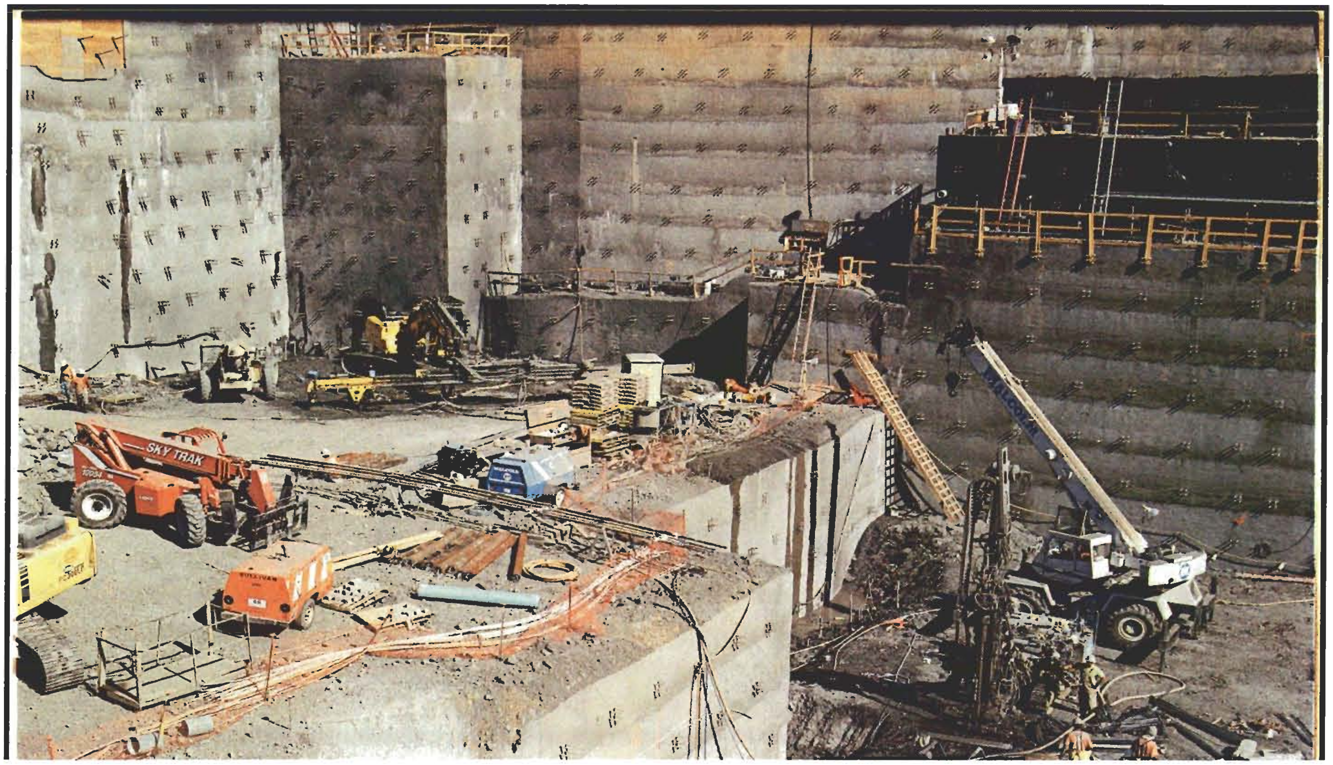
Method of Analysis

The analysis of soil-nailed systems is typically based on an ultimate strength procedure. This procedure is based on an assumed failure plane and calculating the safety factor against failure. The loads acting on the system consist of soil dead load, surcharge due to adjacent building, or traffic, water pressure and lateral and vertical seismic pressure. The resisting forces include the friction and cohesion of the soil as well as the forces in the nails.

The failure plane is assumed a bilinear surface, and various planes shall be considered during analysis to find the one with the lowest safety factor. As the soil can contain several layers with different properties, it is important to consider the variation of soil layers.

A computer program, Winslope was developed by Dr. Toorak Zokaie, P.E., for PB&A and was used to carry out this computational effort. It is noted that the desired safety factor for the three cases of temporary, permanent static, and permanent dynamic (seismic) are all different. Furthermore, the cohesion and friction of the soil during seismic condition can be very different than those of the static condition. These variations must be taken into account during the analysis process. A general overview of the analysis as used in Winslope program is given below.

Fig. 5: Northeast corner of excavation



Winslope Analysis

The basic analysis assumption is that the failure condition is made of two wedges. It is assumed that the two wedges (as shown in Figure 6) fail with a vertical plane. Another assumption is that the lower wedge (wedge 1) will move toward the outside and the upper wedge (wedge 2) will move downward. The forces acting on the total system include weight (W), Nail forces (N), Friction (Fr), Cohesion (C), and Normal force at the intersection (R). Considering Wedge 1 and Wedge 2, the following forces will act on the system, as well as on the interface between the two wedges. Note that the failure surface is called ABC, where A is the toe, C is on the surface, and B is the interface between the two surfaces. Additionally, the location on the surface directly above point B is called 'B'.

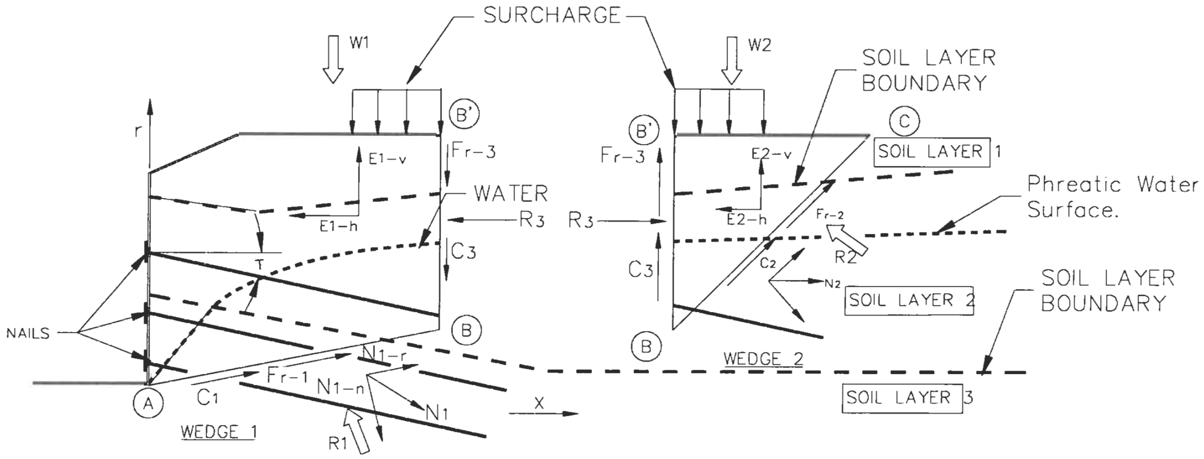


Fig. 6: Freebody Diagram

Note that these forces are all known with the exception of the interaction forces (R1, R2, and R3). The safety factor is defined as the factor that can be used to reduce the resistive forces (Friction, Cohesion, and Nail) to equate the driving forces (Weight, Earthquake, and Interaction, R). Considering the three unknown forces and the unknown safety factor, there are a total of four unknowns. Using the equilibrium equations for forces on each wedge in horizontal and vertical directions, four equations are available to solve for the four unknowns.

The solution scheme considers the equilibrium of each wedge, and the fact that it has three unknowns. For example, Wedge 1 has R1, R3, and f (safety factor) as unknowns. Assuming a value for the safety factor (f) the other two can be obtained. Therefore, a value for R3 is calculated. The same is repeated for Wedge 2, and another value for R3 is obtained. The correct solution is obtained when the two values of R3 are the same. This is obtained by trial and error (iterations), i.e., different values of f are tried until the R3 value from Wedge 1 and Wedge 2 are within an acceptable tolerance.

Note that it is possible that in some cases an acceptable safety factor cannot be obtained due to numerical nature of the solution. If after a maximum specified number of iterations, a solution is not reached, it usually means that the safety factor is too low, and the solution should be revised by providing more resistance, usually achieved by providing more nails.

Analytical Modeling

One of the issues complicating the analysis of the Stanley Hall site is the fact that the excavation is not always on a straight wall, but has several layers and steps. This requires that assumed failure planes consider a failure of the entire wall as well as local failure of the lower stepped walls.

Due to the variations in site and soil conditions around the perimeter of the excavation, several analysis models were created to capture the behavior of each location.

Table 1: List of Forces acting on the Slope

Notation	Description	Direction	Act on Wedge	Condition	D/R*
W1	Weight of Wedge 1	Vertical (downward)	1	Known	D
N1-N	Total force from all nail (Component)	Normal to edge AB	1	Known	R
N1-P	Total force from all nail (Component)	Parallel to edge AB	1	Known	R
R1	Normal interaction force along AB	Normal to edge AB	1	**Unknown**	D
Fr1	Friction force	Parallel to edge AB	1	Known	R
C1	Cohesion force	Parallel to edge AB	1	Known	R
E1-H	Earthquake force	Horizontal	1	known	D
E1-V	Earthquake force (Uplift)	Vertical (upward)	1	known	D
R3	Interaction force along BB'	Normal to edge BB'	1&2	**Unknown**	D
Fr3	Friction force	Parallel to edge BB'	1&2	Known	R
C3	Cohesion	Parallel to BB'	1&2	Known	R
W2	Weight of Wedge 2	Vertical (downward)	2	Known	D
N2-N	Total force from all nail (Component)	Normal to edge BC	2	Known	R
N2-P	Total force from all nail (Component)	Parallel to edge BC	2	Known	R
R2	Normal interaction force along AB	Normal to edge BC	2	**Unknown**	D
Fr2	Friction force	Parallel to edge BC	2	Known	R
C2	Cohesion force	Parallel to edge BC	2	Known	R
E2-H	Earthquake force	Horizontal	2	known	D
E2-V	Earthquake force (Uplift)	Vertical (upward)	2	known	D

D = Used Directly, R = Reduced by safety factor

One of the most critical sections is the southern portion of the east wall as shown in figure 5. This location requires a 77 ft. deep excavation including a 20 ft. wide step at 53 ft. high. The soil is made of three layers of fill, native soil and shale. A 20 ft. wide planter at the top of the excavation exerts 630 psf of surcharge, and the roadway, 50 ft. away from the top of wall, exerts a 300 psf surcharge. The water table is 15 ft below the top of the wall adding to the instability of the excavation.

The Winslope analysis model is shown in Figure 8. The entire wall geometry was modeled in one analytical model, checking different depths of excavation including temporary and final cases. This site was modeled for three different conditions considering temporary excavation, final condition, and seismic loading. The seismic load was modeled as 25% the static loads, including surcharge.

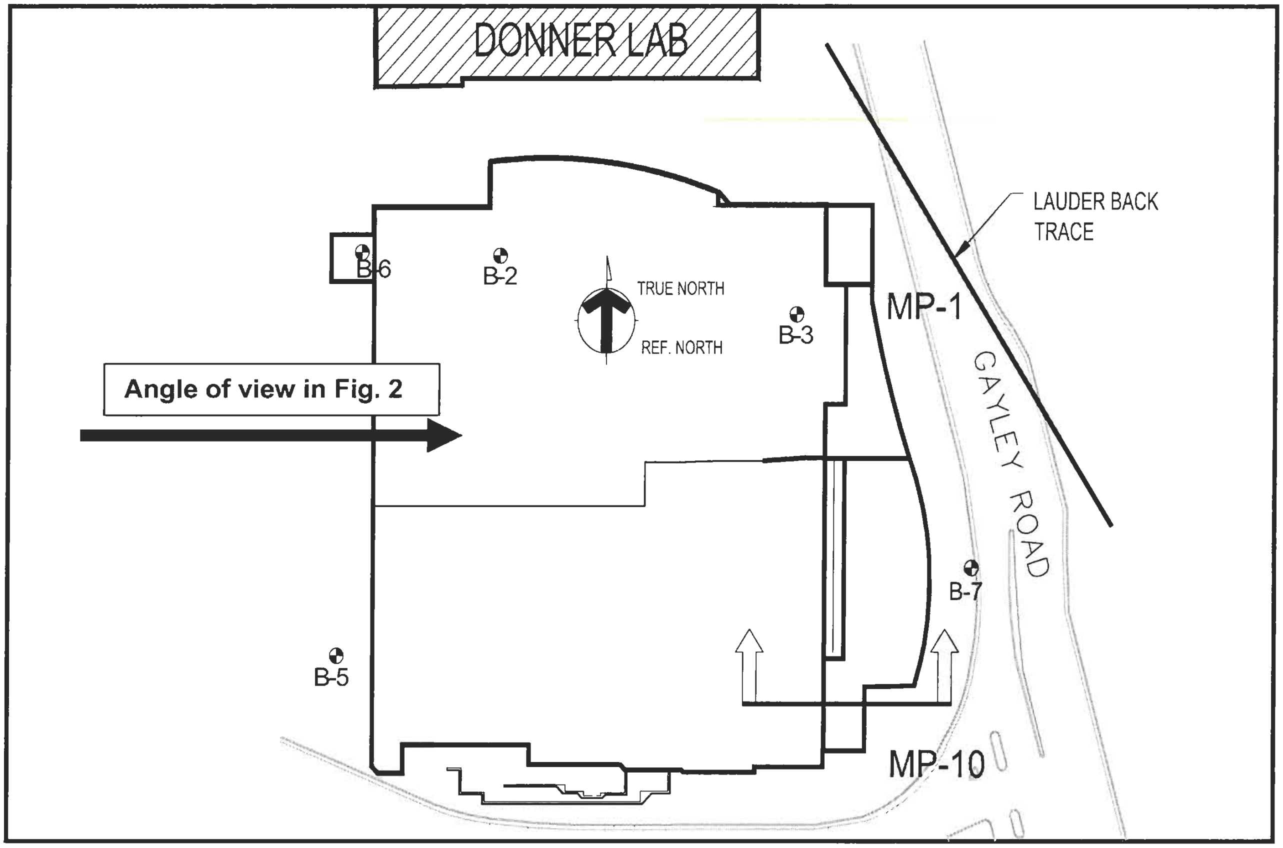


Fig. 7: Plan View showing Model Location

The nails were installed in 8" diameter drilled holes and were high strength (150 ksi), 1.25" diameter threaded bars spaced at 6 ft. horizontally. The length of the nails varied from 50 ft. at the bottom to 80 ft. at the top half of the wall. The deep excavation and long nails require that the solution boundary be extended to a long distance beyond the top of the wall. The critical failure surface was found to cross the soil surface at 20 ft., 40 ft., and 90 ft. for cases of temporary, permanent and seismic respectively; see figure 6. The respective safety factors for the same cases were 1.43, 1.53, and 1.24.

Drilling of 80' long nails at the top elevations of the wall also presented a challenge to the contractor,

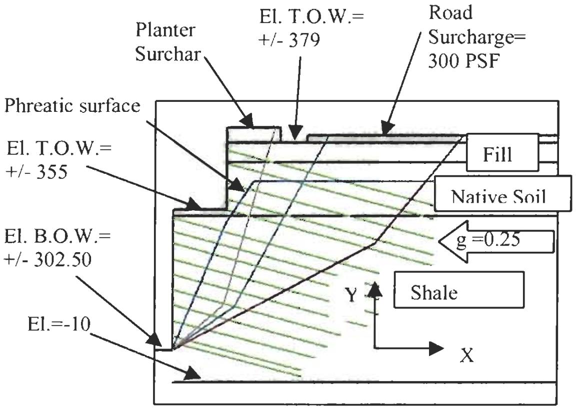


Fig. 8: Wall Model at South-East Corner

DrillTech. With the 15° inclination an 80' long nail would drop about 20' and in variable soil conditions of this site would go through various soil strata which required different drilling bits (going from softer shale to hard greywake) though the holes would invariably stay open.

Drainage, Water Proofing, Corrosion Protection

In order to ensure a successful permanent Soil Nail wall integration into the structure of a building, there are three important elements that must be taken into consideration, and they are inseparable. These three elements are drainage, water proofing and corrosion protection.

Fig. 9: Nelson Stud

Fig. 10: Recessed Shear Plate

In the case of Stanley Hall, because of the very high water table, drainage and water proofing were of predominant importance. Also, the fact that the structural wall and the Soil Nail wall were to be integrated by "stitching" them together through the use of Nelson Studs (see Fig. 9), the design called for the shear plates to be recessed into the exposed face of the excavation (see Fig. 10), allowing only the Nelson Studs to protrude through the water proofing surface.

Corrosion protection was achieved by encapsulating double corrosion-protected Soil Nails (DCP) in polyethylene corrugated sheathing. The tips of the Soil Nails, as well as shear plates and miscellaneous attachments, including the Nelson Studs, were epoxy coated. See Fig's. 9 and 12.

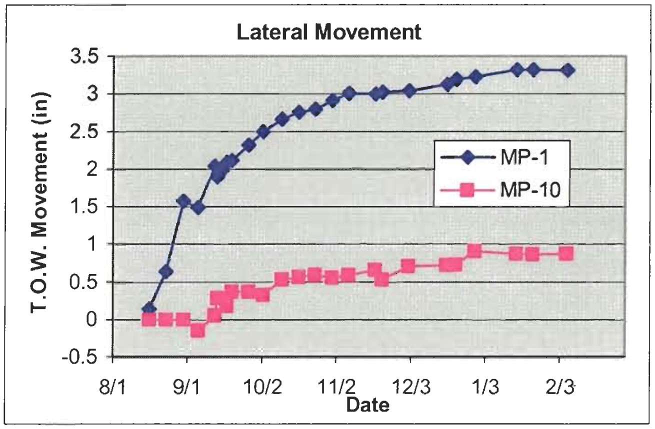


Fig. 11: Horizontal Movement – MP-1 & MP10

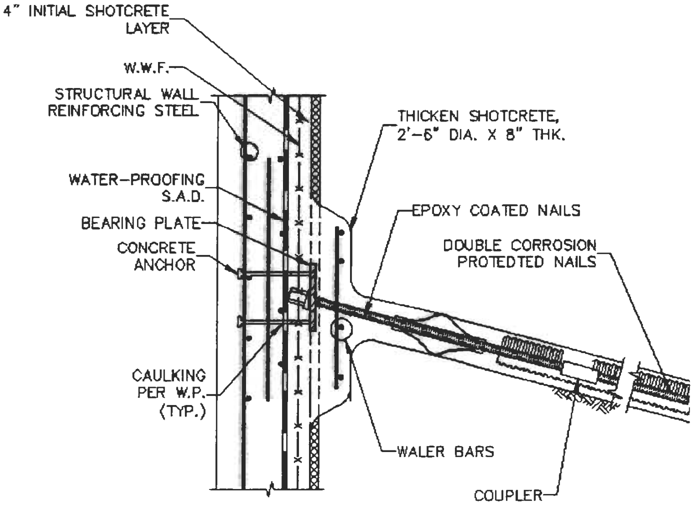


Fig. 12: Detail of typical Soil Nail

Plaxis

Two dimensional Finite Element Analysis Program "Plaxis" has been used to corroborate the wall deformations and to calibrate the parameters used in the design. The objective was to choose soil parameters and moduli for different strata so that the deformations of the wall would duplicate the actual field measurements. Further, by method of $\varphi - C$ reduction, a Factor of Safety was found that was compatible with the Factor of Safety calculated using "Winslope".

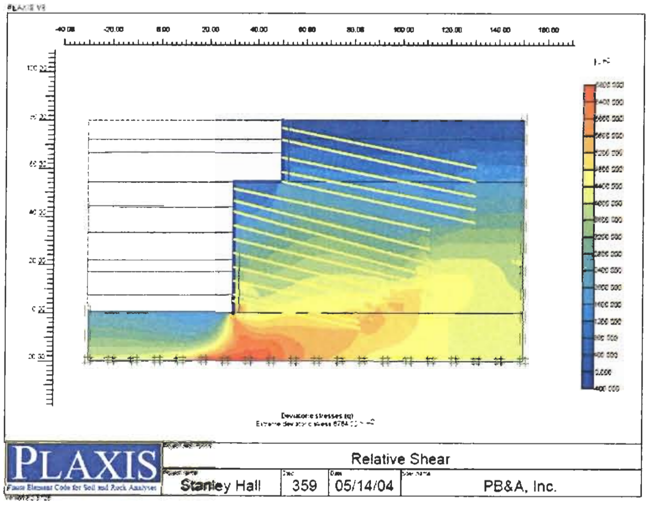


Fig. 13: Relative Shear Stress

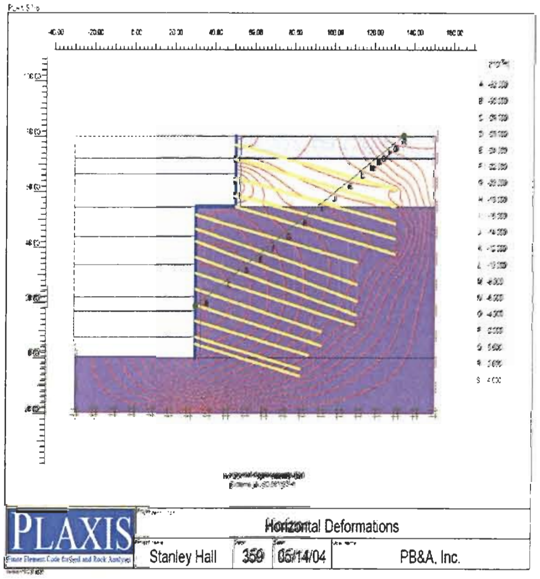


Fig. 14: Horizontal Deformation Contours

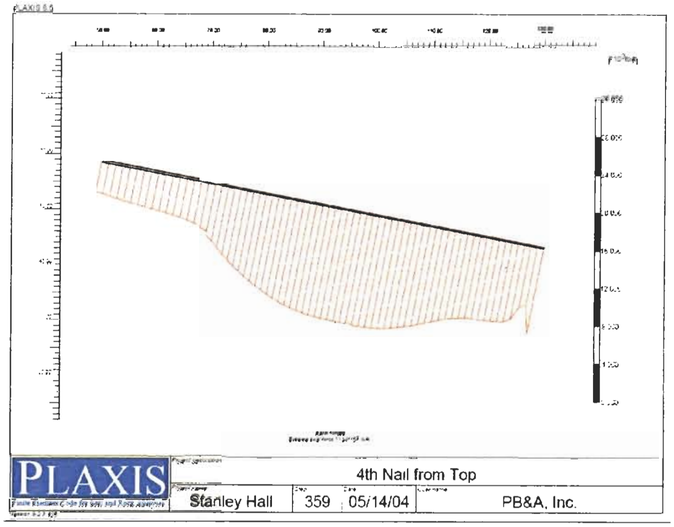


Fig. 15: Stress Distribution in 3^{rd}. Row Nail

PB&A has been using Plaxis as a collateral design tool for almost four years. The final goal is to accumulate enough data and experience so that Plaxis will become the main design tool used by PB&A. In order to achieve that goal, parallel analyses such as this is being conducted, to not only compare results with the orthodox analytical techniques, but also with the actual data gathered in the field.

Monitoring

The nail force is mobilized as the failure plane starts to form and creates tension in the nails. Therefore, it is important to monitor the movement at the top of the wall to make sure that the movements are within acceptable ranges and that settlement and lateral movements stabilize as the nails are activated. The acceptable movement is a function of the height of the wall, with upper bound of 0.5% of the height.

Another reason for monitoring is to assess the movement, and if excessive or too rapid, provide corrective measures. Monitoring stations were set up at various locations around the Stanley Hall site, and vertical, lateral, and parallel movements were measured. Typical movement at top of wall is in the order of 0.1% to 0.2% of the wall height. This translates to 1 to 2 inches for a 75 ft high wall.

Monitoring stations MP-1 and MP-10 are located at northeast and southeast along the east wall. Although these walls have similar heights, they exhibited different amounts of movement. Station MP-10, which is close to the location were analysis model was presented, had less than 1 inch of movement as shown in figure 7. Generally,
locations near corners are fairly stable and do not show excessive movements, but due to the height of excavation and geology of the site, the northeast corner of this site showed a few inches of lateral movement; see figure 9.

PB&A, Inc. monitored this movement and determined that the time lapse between excavation of a layer and installation of the nails needed to be shortened to limit the movement. The excavation started in mid-August, 2003 and continued until end of January, 2004. During the first half of September the construction procedure was revised to accelerate the installation of the nails. Although the excavation continued for several months after this time, the rate of movement was reduced drastically, and eventually stabilized.

Conclusions

The Stanley Hall construction, with 80 ft. deep excavation, varied geological conditions and topography provided a challenge that was met by use of soil-nailed wall construction. Over 75 ft. deep excavation with several steps, high surcharge loads, and various soil layers created a situation that required detailed analysis. Since these walls are permanent, corrosion protection and monitoring were made an integral part of the design.

This project proves that soil-nailed systems can be used to overcome challenges that are encountered in deep excavation situations. Furthermore, soil-nailed systems can be used effectively for permanent as well as temporary construction, are often an economical choice. The experience of the designers in mitigating the design and construction challenges is the key to a successful project.

References

1. Felio, G.Y., Vucetic, M., Hudson, M., Barar, P., and Chapman, R., 1990, "Performance of Soil Nailed Walls During the October 17, 1998 Loma Prieta Earthquake," IN Proc. Of the 43rd Canadian Geotechnical Conference, Quebec, Canada, pp.165-173.

2. U.S. Department of Transportation, Federal Highway Administration (FHWA), 1996, "Manual for Design & Construction Monitoring of Soil Nail Walls," FHWA, Publication No. FHWA-SA-96-069

3. U.S. Department of Transportation, Federal Highway Administration (FHWA), 1996, "Soil Nailing Field Inspector's Manual," FHWA, Office of Technology Applications, Publication No. FHWA-SA-93-068