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File: 201071
May 31, 2001

Avalon Bay Communities
4340 Stevens Creek Blvd., Suite 275
San Jose, CA 95129

Attention: Nathan Hong

Subject: **Avalon Bay at Lake Merritt**
230 Bay Place
Oakland, California
GEOTECHNICAL INVESTIGATION
FOR PROPOSED DEVELOPMENT

Dear Mr. Hong:

In accordance with your authorization, we have performed additional subsurface investigation services at the project site in order to enable our engineers to provide design and construction recommendations for the development proposed at the site.

Our supplemental subsurface exploration program has verified that there are soft deposits of compressible soil materials located towards the northern third of the site (area towards the intersection of Bay Place and Harrison Street). These materials appear to have been placed predominantly by natural processes as an old stream channel which previously fed into Lake Merritt was filled by soils washing down off the hillsides. However, some man-placed fills have been placed over the top of those softer natural soils, apparently to create the level building pads for the buildings which currently occupy the site. Shallow ground water (less than 5 feet deep) was also encountered at the site. Fortunately, soils on the southern portions of the site, including on the slopes to the south and east of the level building pad were found to be quite strong.

Based upon our supplemental investigation, we have reached the following opinions:

1 - deep-seated failures of the rear slopes (below the adjacent residential buildings) are unlikely to occur during replacement of the existing retaining walls and associated grading work;

2 - about two-thirds of the new building can be supported by conventional spread footings, while the northwestern portion of the building will need to be supported by either cement-treated-soil columns or mat foundations so as to limit differential settlements within the structure, and those which might be induced to the remaining Cox Building;

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3 - seasonal ground water will be encountered in the sub-slab granular section and should be collected by a sub-slab drainage system (contact Levine-Fricke for a discussion on need to remediate the collected waters);

The attached report discusses the work we performed, presents our findings and conclusions on the geotechnical conditions, and provides our recommendations for the design and construction of the proposed development.

Should you have any questions regarding the information contained in the report, please contact the undersigned.

Respectfully Submitted:
GeoForensics, Inc.

Daniel F. Dyckman, PE, GE
Senior Geotechnical Engineer, GE 2145

cc: 10 to addressee
1 to EQE
1 to FBA
1 to Levine-Fricke

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GEOTECHNICAL INVESTIGATION FOR PROPOSED DEVELOPMENT

at

Avalon Bay at Lake Merritt

230 Bay Place

Oakland, California

Report Prepared for:

Avalon Bay Communities

Report Prepared by:

GeoForensics, Inc.

May 2001

GEOTECHNICAL INVESTIGATION
FOR
PROPOSED RESIDENTIAL DEVELOPMENT

Avalon Bay at Lake Merritt
230 Bay Place
Oakland, California

BACKGROUND

Site Description

The project site is located on the eastern corner of the intersection of Bay Place and Harrison Street in Oakland (see Figure 1). Vernon Street forms the southeastern property border to the site. To the northeast of the site, multi-story apartment buildings occupy the adjacent lots on Vernon Terrace. A set of concrete stairs and sidewalks extends from these adjacent buildings, down to Harrison Street at the northern corner of the project site. Lake Merritt is located about one block to the west of the subject site.

The site is located at the base of the foothills which define the eastern lateral margins of the Oakland portion of the San Francisco Bay valley floor. The topography of the area consists of a broad drainage swale (under Harrison Street), with moderately to steeply sloping ridge slopes to either side (see Figure 2). The southern side ridge extends down from Vernon Terrace into the subject property, but has been cut at the nose to provide additional level area on the site. The axis of the broad swale under Harrison Street, appears to have previously passed under the western corner of the subject site at the intersection of Bay Place and Harrison Street. That swale has been filled in through both natural processes and the work of man (fill).

Currently, the majority of the subject lot is nearly level, at the elevation of Bay Street. The remaining northeastern portion of the project site consists of a structurally supported vertical cut (retaining wall is up to about 20 feet tall) at the base of a very steep slope which extends up about 60 feet (vertically) to the apartment buildings on Vernon Terrace. The slopes above the retaining walls range from about 2:1 to as steep as 1:1 (see Figure 4). The slopes are vegetated with native grasses, bushes, and some large trees.

Asphalt parking surfaces cover the northern corner of the property, and the southern end of the lot. The central portion of the lot, over to Harrison Street and Bay Place, is covered with commercial buildings, previously used as a car dealership and service facility. The front dealership building (the Cox Building) was constructed with large windows between block/brick columns along Harrison Street and Bay Place, and concrete block/brick walls along the back of the building. Interior walls and ceilings are plaster surfaced.

The rear buildings consist of steel-framed open structures along the Harrison Street side, and concrete perimeter walls with concrete interior columns in the eastern portions of the buildings (adjacent to the rear cut slope).

Floors in all of the buildings consist of concrete slabs-on-grade. The concrete slabs in the western end of the Cox Building are noticeably out of level, with depressions and ridges (see site observations below).

Surface drainage at the site appears to be by surface flow off the roof areas and pavements to the storm drain system in the adjacent streets. Some catch basin facilities were observed inside the building, but these are more likely tied to other drainage facilities for discharge.

Project Description

The project is to commence with the demolition of several existing commercial buildings on the subject site. Only the Cox Building, an approximately 60 wide by 200 foot long structure, facing onto Bay Place will remain on the subject property.

The Cox Building will be renovated for commercial retail space. As part of the renovation, the existing building is to be seismically upgraded to current codes. According to EQE, the project seismic engineers, this will require the installation of new strip footings along Harrison Street and Bay Place, as well has new isolated footings at column locations along the rear (western end) of the building. Structural dead loads between 10 and 15 kips are anticipated, with seismic loadings of 10 to 15 kips (upwards and downwards) on the strip footings, and 90 to 105 kips on the isolated footings.

In the remaining portions of the site (the area where the existing structures are demolished, and in the areas currently occupied by asphalt pavements) a new 7-story residential structure is proposed. The proposed structure is to consist of two stories of concrete-framed parking garage, with up to 5 stories of residential, wood-frame construction above the parking garage podium. The ground story level of the parking garage is to be depressed about 3 feet below current grades, and to consist of concrete slab flooring/pavement. Structural loads at the columns are to be as much as about 450 kips at some locations.

New and replacement retaining walls, with heights up to 25 feet, will be required at the back of the project site to support the base of the existing steep slopes (below the residential structures on the adjacent lots to the west).

INVESTIGATION

Scope and Purpose

The purpose of our investigation was to refine our understanding of the nature of the subsurface soil conditions originally identified by Lowney Associates, so that we could provide final geotechnical recommendations for the construction of the proposed new 7-story residential structure, and the foundation renovation work for the Cox Building. In order to achieve these purposes, we have performed the following scope of work:

1 - visited the property to observe the geotechnical setting of the area to be developed;

2 - reviewed relevant published geotechnical maps;

3 - reviewed the preliminary reports by Lowney Associates;

4 - drilled 8 additional borings at various locations across the site;

5 - performed laboratory testing on collected soil samples;

6 - met with your structural engineers to discuss project needs;

7 - assessed the collected information and prepared this report.

The findings of these work items are discussed in the following sections of this report.

Site Observations

We visited the site in April and May, 2001 to observe the geotechnically relevant site conditions. During our visit, we noted the following conditions:

A - The floors in the existing Cox Building near Harrison Street have settled relative to the perimeter and interior foundations. The unlevel floors are apparent as a droop between the footings in the two western-most "bays" of the building. In the eastern two-thirds of the building, the floors appear to be relatively level.

B - A test pit excavated for EQE near the front of the Cox Building exposed a 3 to 4 inch void extending under the demolished section of slab. The void could be seen to extend back towards the center of the slab area for a distance of several feet. At the edge of the excavated pit, a brick footing was observed to extend from the front foundation of the building, back under the slab towards the rear of the building. The slab was higher in elevation over the top of the footing.

C - The test pit at the front of the building also demonstrated that the existing Cox Building foundations consist of brick and mortar. The footings extended over 5 feet below the top of slab elevation, deeper than the hole was advanced. Ground water had accumulated in the hole to a depth of 5 feet below grade, destabilizing any further excavation in the loose clean sands exposed in the pit walls.

D - Another test pit, excavated along the back wall of the Cox Building, also exposed a brick and mortar foundation under that wall. The brick footing extended over 4 feet below grade. Materials exposed around the footing appeared to consist of soil and rubble fill. Ground water was also present in this hole at a depth of about 5 feet below top of existing slab.

E - Some cracking of the block wall along the back of the Cox Building was observed from within the adjacent shop facility. The cracking was noted in the wall at the peak of the roof over the door between the showroom and the shop. This cracking suggests that there may have been some previous settlement of the block wall foundation. This is in the area where the Lowney subsurface exploration, and our exploration, identified soft alluvial soils in the old creek channel (see following sections of our report).

F - A shallow slope failure was observed on the steep hillside at the back of the project site. The slump is approximately marked on Figure 4 (site plan). The slump is estimated to be less than 5 feet thick, but should be removed during construction of the new retaining walls.

G - The slope on the subject property to the northwest of the Smith property (Lot 12 on Vernon Terrace) is overly steep, with a slope gradient on the order of 1:1. Lateral movements of the foundation for that building appear to have occurred in the past, with a more recent concrete wall/footing installed along the side of the house. However, even that footing/wall can be seen to be moving laterally downslope. It would appear reasonable to flatten this slope to about 2:1 to provide better lateral stability to the adjacent property.

H - The property has been used for many years in an industry associated with gasoline, oil, cleaning agents, and other chemicals associated with ground water and soil contamination. Potential contamination and other impacts on the construction are being evaluated by Levine-Fricke for this project.

Map Review

We reviewed the *Preliminary Geologic Map Emphasizing Bedrock Formations in Alameda County, California: Derived from the Digital Database Open File 96-252* by Graymer (1996). The relevant portion of the Graymer map has been reproduced in Figure 3. This map indicates that the site is underlain by Undivided Surficial Deposits (map symbol "Qu"), indicating that the depth to bedrock is likely to be quite deep.

Our subsurface exploration (see below) encountered various soil materials consisting of man-placed fill, soft alluvial deposits, and stiff colluvial deposits to depths over 40 feet, consistent with the geologic mapping.

The active Hayward Fault is mapped approximately 3 miles northeast of the site, the Calaveras Fault 12 miles northeast of the site, and the San Andreas fault approximately16 miles southwest of the site. All of these faults are capable of producing strong ground accelerations at the subject site. No faults are mapped on, or trending towards, the subject property.

Lowney Associates Reports

August 4, 2000 - Lowney Associates (Lowney) issues a "Draft Fax Transmittal" providing the logs of the borings and CPT soundings conducted on the site in late July. Lowney identified "differing shallow soil conditions over the eastern half and northwest quarter of the site, and the southwestern quarter of the site". On the eastern half of the site, the borings and CPT testing found about 3 feet of fill over stiff to very stiff clays and sandy clays. In the southwestern portions of the site, EB-1 and CPT-1 penetrated the same 1.5 to 3 feet of fill, but is was underlain by soft to medium stiff clay soils to depths up to 16 feet. Under the soft materials, these borings encountered similar stiff to very stiff clays and sandy clays to those encountered in the eastern half of the site. Ground water levels were not measured, but were estimated to be between 0.1 and 10 feet in the various soundings, and reported to be on the order of 2 to 5 feet from discussions with other consultants. The logs from these borings and soundings are attached in Appendix C, and their locations are shown on Figure 4.

August 8, 2000 - Lowney prepares a fax report discussing their "Preliminary Geotechnical Findings" for "Building Foundations". It was their opinion that the proposed construction was possible, but that there were four primary geotechnical concerns related to the construction, including: 1) the presence of shallow ground water; 2) the close proximity of the adjacent streets and buildings to the proposed excavations; 3) the high expansion potential of the native near-surface soils; and, 4) the presence of site fills and disturbed soils.

To address the high ground water, Lowney stated that a sub-slab drainage system would be required, or that the slabs would need to be designed to resist hydrostatic uplift. The high water would also create a need to dewater during construction. The excavations close to the City streets would require shoring to be installed, while the replacement of the rear wall was to be addressed in a separate letter (see below). To address the expansive soils, a 12 to 18 inch layer of non-expansive select fill was to be placed. The fill could consist of lime treated native clay soils, as it would also provide a good working platform. Finally, the disturbed soils and much of the fill was to be removed as part of the proposed excavation to lower the ground story slabs, however, any remaining fill or disturbed soil was to be removed down to expose native soils within the building footprint.

Based upon the above concerns being properly addressed, Lowney recommended that the foundations for the building could consist of spread footings embedded 5 feet below grade. An allowable bearing pressure of 2 to 3 ksf was proposed for footing sizing. Settlements were estimated to be on the order of 1.3 to 1.7 inches under the anticipated loads of 432 kips per column. Differential settlements between similarly loaded columns was to be on the order of 0.5 inches. However, these recommendations did not apply to the western third of the lot where the soft soils were found. In that area, Lowney recommended ground improvements be made using soil mixing down to depths on the order of 16 feet. Alternatively the columns could be integrated into a structural mat foundation. A differential settlement of 1.5 inches was projected between interior and corners of the mat.

August 8, 2000 - Lowney issues a second fax report on their Preliminary Geotechnical Findings regarding the "Retaining Walls and Slopes". In this fax, Lowney expresses concern regarding the temporary stability of the cut slope at the back of the project while the 20 foot high existing retaining wall is demolished, and a new 25 foot tall wall is constructed. Based upon assumed ground strength parameters, Lowney calculates that the factor of safety against rotational failure of the slope is below 2.0, the minimum value considered to be appropriate without better subsurface information. It was recommended that further investigation be conducted to reduce uncertainty regarding the soil strength parameters, thereby allowing for a lower minimum factor of safety to be used (1.5). Calculated factors of safety for the various strength parameters were: ϕ=32, FS=1.23; ϕ=34, FS=1.34; ϕ=38, FS=1.56. No cohesive strength was assumed for this analysis along most of the slip surface (hence a conservative analysis).

March 21, 2001 - Lowney identifies the scope of additional work which they believe is required to complete the geotechnical analysis of the site. Their study is to be aimed at: 1) further delineation of the extent of the compressible soils at the western corner of the site; and, 2) determination of the soil conditions at the base of, and above, the proposed new tall retaining walls.

Subsurface Exploration

After reviewing the Lowney reports, we concurred that additional subsurface investigative work was required to better define various aspects of the site soil conditions. Therefore, on April 5, 2001 we drilled six (6) new borings at the site at the locations shown on Figure 4. Borings GF-1 through GF-5 were drilled using a Mobile B-24 truck-mounted drilling rig, while Boring GF-6 was drilled using a Minute Man portable drilling rig. The rigs were equipped with 4.0 and 3.25 inch diameter, helical flight augers, respectively.

Subsequent to the drilling to augment the Lowney study, we were contacted to provide geotechnical recommendations for the Cox Building. As no available subsurface information existed within the Cox Building, we drilled another 3 borings, GF-7 through GF-9, in the Cox Building at the locations shown on Figure 4. These borings were all drilled using the Mobile B-24 truck-mounted drilling rig.

Logs of the soils encountered during drilling record our observations of the cuttings traveling up the augers and of relatively undisturbed samples collected from the base of the advancing holes. The final boring logs are based upon the field logs with occasional modifications made upon further laboratory examinations of the recovered samples and laboratory test results. The final logs are attached in Appendix A.

The relatively undisturbed samples were obtained by driving a 3.0 inch (outer diameter) Modified California Sampler and a Standard Penetration Sampler (as noted on logs) into the base of the advancing hole by repeated blows from a 140 pound (truck rig) and a 70 pound (portable rig) hammer lifted 30 inches. On the logs, the number of blows required to drive the sampler the final 12 inches of the 18 inch drive, have been recorded as the Blow Counts. These blows have not been adjusted to reflect equivalent blows of any other type of sampler or hammer, or to account for the different hammers and samplers used.

Subsurface Conditions

Avalon Bay Building Site - Boring GF-1, drilled near the base of the 20 foot high concrete retaining wall at the back of the property, first encountered concrete from the footing. The hole was moved away from the wall and was redrilled. We estimated that the footing is on the order of 10 to 12 feet wide. Under a surficial layer (2 foot thick) of gravelly sandy clay, the natural soils consisted of sandy clays and clayey sands in a very stiff to medium dense condition to the base of the boring at a depth of 20 feet. Ground water was first encountered at a depth of about 8 feet, but after a few hours, the water was observed to be seeping up out of the hole, indicating that an artesian condition exists in the sandier layers below grade.

Borings GF-2 through GF-4 were drilled in the rear service building in an effort to better define the zone of soft soils first identified in the Lowney Associates field investigation. These borings generally encountered a thin veneer of materials judged to be soft fills, over a soft black or green-grey clay. The soft clays extended to depths of about 8 to 12 feet below grade, where the soils tended to contain greater amounts of sand, changed to a orange or brown color, and were in a stiff to very stiff condition. By a depth of about 15 to 20 feet below grade, the soils stiffened even further to be classified as very stiff. Ground water was generally encountered in the soft clay and sandy layers, and was measured at a depth of about 6 to 12 feet below grade prior to backfilling the borings. However, we do not believe that this represents a stabilized ground water level.

Another boring (GF-5) was attempted within the rear service building, however, this boring was terminated in thick concrete. The boring easily penetrated the floor slab to the building, then penetrated about 3 feet of sandy clay fill before it reached another thick section of concrete. The boring was drilled to a depth of 6 feet below grade, but did not extend below the base of the concrete. The hole was relocated about 5 feet away, but the same conditions were again encountered. No ground water was noted in this aborted boring.

The final boring (GF-6) drilled on April 5, was drilled at the top of the slope behind the Smith property (Lot 12 on Vernon Terrace). This boring penetrated about 7 feet of dark brown, firm to stiff sandy clay/clayey sand colluvial soils. From 7 to 16 feet, the sand content increased and the soil changed to a orange-brown color, becoming medium dense to very stiff. Very stiff sandy clay was then encountered between 16 and 19 feet, before the boring was terminated in a dense silty fine sand at a depth of 19.5 feet. No ground water was encountered in this boring.

Cox Building Site - Borings GF-7 and GF-8 were drilled in the Cox Building to define the nature of the site soils in the area of the proposed new seismic strip footings, and to further delineate the nature of the soft soil zone which passes through this corner of the site. The borings encountered about 5 feet of fill, including loose sands and a 3 foot thick section of concrete. Under these fill materials, dark grey soft alluvial materials were present to depths of about 13 to 17 feet below the slab surface. Very stiff to hard, orange-brown colluvial materials were then penetrated to the base of the borings at 19.5 feet. Ground water was measured at a depth of 5 to 7 feet below grade.

Boring GF-9 was drilled at the location of a sampling hole excavated by Levine-Fricke. The upper 3 feet consisted of layers of concrete slab and fill sand. From 3 feet to a depth of 12 feet, the materials were already excavated by Levine-Fricke prior to our drilling. Reportedly, the soils tightened at a depth of about 8 feet. Our drilling and sampling started at a depth of 12 feet and extended to 14.5 feet. The materials we encountered consisted of a very stiff, orange-brown sandy clay. Ground water was reported by Levine-Fricke at a depth of 5 feet below grade.

These new borings were pre-cut through existing concrete materials in order to permit drilling. In most cases, this resulted in the disturbance of the upper 5 feet of soils and concrete prior to our arrival. Therefore, our logs of materials generally start at a depth of about 5 feet below grade.

Two test pits excavated for the EQE preliminary study indicate that the upper 5 feet of soils under the Cox building consists of sands, often with interior brick footings, or concrete slabs (up to 3 feet thick) embedded at various locations. The sands appear to be in a loose condition, as they have settled with respect to the deep brick footings which were observed, leaving a void under various portions of the slabs, and allowing sections of the slabs to settle. This differential suggests that the relative drop of the slabs has developed due to loose conditions in the upper 5 feet of soil, rather than due to deep movements of the underlying soils. We have requested, but have not received, a survey of the building floors in order to assess whether the deeper footings have also been subject to movement in the past.

Please refer to Appendix A for a more detailed description of each boring.

Laboratory Testing

The relatively undisturbed samples collected during the drilling process were returned to the laboratory for testing of engineering properties. In the lab, selected soil samples were tested for moisture content, density, strength (direct shear), consolidation characteristics, and Atterburg Limits. The results of the laboratory tests are attached to this report in Appendix B.

Strength testing was conducted on the several soil samples, including: the very stiff sandy clay soils on the eastern portions of the pad (at the tall rear retaining wall); over and under the soft alluvial soils on the western portions of the pad; and, the stiff soils on the slope above the rear retaining walls. In general, the stiff sandy clay soils on the slope and in the eastern portions of the site were quite strong. Conversely, the softer soils in the western side of the lot were relatively weak, though they increased in strength significantly with depth.

The Atterburg limit testing found that even the deep soils have a high plasticity index (46). This indicates that the deeper soils are also expansive, and would not provide a good source for structural fills. Previous work by Lowney reported the soils to have high expansion potential.

Consolidation testing was conducted on seven soil samples to determine the potential settlements of the various different stiffness soils under loading. Both constant rate of strain testing and conventional incremental loading testing was conducted. The testing indicated that the softer soils are lightly over-consolidated, while the deeper soils are moderately to highly over-consolidated. Over-consolidated soils will tend to settle less under loads up to their pre-consolidation pressure, while normally consolidated soils and pressures greater than the pre-consolidation pressure can cause greater settlements.

DISCUSSION

Foundation Considerations - Cox Building

From our discussions with EQE, we understand that the seismic retrofit work is to consist of a new continuous footing along the front (Bay Place) and northern (Harrison Street) side of the building, and the installation of isolated column footings along the rear (block wall) side of the structure. Dead loads for the continuous footings are expected to be on the order of 15 kips (per column location) with seismic loads between 10 (downward) and 15 (upward) kips. The isolated footings would also have dead loads of 15 kips, but seismic loadings of 105 (downward) and 90 (uplift) kips.

Access for large equipment inside the Cox building would require partial demolition of the structure, which will not be permitted due to the historic nature of the building. However, some portable equipment of limited size may be able to access the building interior.

Additionally, the Cox Building is constructed of 100 year-old plaster, brick, and mortar which will preclude the new or repair construction from inducing significant building vibrations (such as from pile driving equipment), or inducing significant post-construction ground settlements.

With any support system, post-construction settlements will likely be on the order of about 1 inch. Therefore, we have set this maximum limit to apply to all ground settlements proximate to the Cox building. This includes settlements of the foundations for the new adjacent Avalon Bay complex, which will have foundation loads imposed to the soils within about 5 feet of the existing Cox building foundations. Therefore, settlements of the ground surface within 20 feet of the Cox building due to any new construction, or retrofit work must be 1 inch or less.

Rear Isolated Foundation Design Options - We have considered two basic design approaches for addressing the back foundation of the Cox building retrofit work:

Option #1 would be to structurally isolate the Cox foundations from the Avalon Bay foundations.

Option #2 would be to structurally connect the Cox and Avalon Bay foundation systems.

The choice of Option #1 was evaluated and determined to be feasible, however, it requires a second set of foundation elements, which do not provide any significant benefit in design for the Avalon Bay foundations (i.e. settlements of that foundation system must still be limited to 1 inch).

The second Option (#2) would allow for the construction of a single integrated foundation system where the high uplift and downward forces from the seismic resistance of the Cox building roof frame can be accommodated within the frame work of the proposed large foundations for the Avalon Bay foundations.

Based upon the relatively easier design and construction, combined with the lack of duplication of design and construction efforts, we are recommending that Option #2 (structural integration) be used for analysis and design of the isolated rear foundation elements for the Cox building.

Front and Side Continuous Foundation Options - The soils which would support the new foundation elements along the side and front of the Cox building have been supporting those same loads since the building was originally constructed. As there will not be any new long term loads imparted to the soils, there should not be any significant settlements of the new strip footings on the existing soils. However, in order to by-pass the loose near surface sandy fill soils, it would be necessary to install these new strip footings at a depth comparable to the existing foundation system in the Cox building, which is on the order of 5 to 7 feet below grade. Such construction would put the foundations at, or slightly below the water table, in an area of loose sand. This could make construction rather difficult.

As an alternative to the use of a continuous spread footing, deep foundation elements may be used to provide vertical support for new footings bearing at a more shallow depth. The new deep foundation elements would need to consist of Chance Augers (Helical piers) used to support the new strip footing (now called a grade beam). As these are proprietary construction elements, the design and construction would need to be performed by the manufacturer's representative. We anticipate that helix piers ranging from about 20 to 30 feet deep would probably result from the design work, depending upon plate size. We would recommend that a conventional "off-the-shelf" plate configuration be used to limit costs. Localized coring or demolition of the deeper concrete materials will be required, where encountered.

Foundation Considerations - Avalon Bay Building

The foundations in the eastern portions of the site will rest on very stiff clayey soils which will provide good bearing with minimal settlements under the new loads. Therefore, the majority of the new Avalon Bay building may be supported upon new spread footings.

However, the western portion of the site is underlain by alluvial deposits extending as deeply as about 15 to 20 feet below existing grades. These soft soils will be subject to settlements under the anticipated loads from the new structure. Those settlements would then be transmitted to the soils at the back of the Cox Building, causing settlements of that structure. Therefore, in order to provide a foundation system which will not damage the Cox building, it will be necessary to limit total settlements in the adjacent portion of the Avalon Bay building. This can be achieved by using a waffle-style foundation with thickening ribs to distribute the high column loads across the entire building footprint, or by providing a deep foundation system which will penetrate through the upper soft soils, to derive support in the underlying stiffer materials.

The deep foundation system proposed in this report, consists of soil-cement columns (termed Geo-Columns or Auger-Piles) created by mixing cement into the soils within a large diameter boring. In this process, a large diameter (30 inch) drill bit is bored down through the soft soils to the bearing stratum. As the auger continues to rotate, cement is forced down through the auger to mix with the soils. The auger is then slowly withdrawn, creating a large diameter column of cement-treated soils (CTS). This CTS column will increase in strength due to the concrete, and then be able to transmit the vertical forces down to the bearing stratum below.

Using this method of foundation support, the settlements imposed on the Cox Building can be minimized, thereby allowing the new seismic loads within the Cox Building to be transmitted to the larger Avalon Bay building for additional support. Total settlements along the Cox Building are anticipated to be less than 1 inch using this approach, with differentials less than 0.5 inches.

Away from the Cox Building, the bearing pressures on the geo-columns can be increased if the resulting differential settlements are not problematic to the Avalon Bay structure.

Shallow Ground Water

Our borings, and those by others have identified that the water table at the site is very shallow, and even above ground surface grades in some locations. It has been our experience that ground water, when under pressure, will tend to find imperfections in most water-proofing systems. It is our expectation that leakage into the lower story of the building will occur if the ground water is not intercepted and drained below the floor slab. Therefore, we have recommended that the non-expansive materials to be provided under the slab should consist of angular drain rock, with collector pipes installed to remove the water. If portions of the site are underlain by contamination (consult with Levine-Fricke regarding any contamination), and ground water treatment will be required, then the underslab drain system may be separated into two or more systems, whereby one would handle clean water, and the other potentially contaminated waters.

Shallow ground water is also likely to make construction access to the project difficult once the overlying concrete slabs have been removed, and as the soils are excavated down the proposed 3 to 4 feet to the pad subgrade elevation. Excavation may require the construction of temporary haul roads stabilized by gravel and fabric, or by lime or cement treatment of the upper materials. It may also require that the excavation be performed by light-weight equipment, or by excavators which can reach out off stable pads. Excavation difficulties are likely to be greatest in the western third of the building pad (where soft soils are present), and possibly at the eastern end of the project where the ground water was encountered at the elevation of the existing ground surface.

Dewatering will be required for the construction of Geo-Column cap foundations and spread footings. It will be necessary for the project environmental consultant to provide recommendations for proper disposal of waters collected by any dewatering system.

Cut Slope Stability

The stability of the rear slope has been evaluated using the strength parameters measured from our laboratory testing of soils on, and at the base of, the steep rear slope. The calculations indicate that the rear slope will possess adequate factors of safety for temporary dry weather construction. Our analysis indicates that the most critical location for potential failure exists along the northward facing slope adjacent to the side of the Smith property (Lot 12 on Vernon Terrace) where a 16 foot high wall will be cut into the slope. Even in this location, the factor of safety against sliding is on the order of 2 for a deep-seated failure.

However, localized sloughing of the near surface soils is possible, particularly where the toe of the shallow existing failure exists, or where loose topsoils are undercut. Therefore, we are recommending that temporary shoring be constructed using cast-in-place piles and tie-backs to allow for stability of the cut face to enable construction of the wall, and stability of adjacent slopes. We have also recommended that the old slide materials be removed from the hillside prior to cutting for the wall.

RECOMMENDATIONS

General

The construction of the proposed new Avalon Bay project, and the Cox Building retrofit work is constrained by loose and soft near surface soils with a shallow ground water table, and on strict requirements on post-construction settlements, combined with high foundation loads. Other constraints may be associated with potential contamination of the soil and ground water (refer to work being conducted by Levine-Fricke for more information on results of their environmental work).

Avalon Bay Building - The settlements of the soils under the Avalon Bay building must be limited proximate to the Cox Building to limit damage to that historic structure. Where new foundation elements for the Avalon Bay building are located more than 20 feet from the Cox Building, foundation settlements may be increased, if the structural engineer deems the degree of potential differential settlements to be appropriate. Construction work on this building will be hampered by shallow ground water and localized soft soils, but will be aided by generally good quality soils over most of the site, and along the slope above the project.

Cox Building - By limiting settlements of the new Avalon Bay building, work on the Cox Building can be simplified by the ability to integrate the large foundation system of the proposed new Avalon Bay building with the isolated foundation elements of the Cox Building to accommodate the anticipated uplift and downward seismic forces in these Cox Building foundations. Construction within the Cox Building is also helped by the pre-consolidation which has occurred under the old foundation elements along the front and sides of the building which would tend to limit settlements of the new foundations.

The foundations for both of these buildings should be designed and constructed in conformance with the recommendations presented in the following sections of this report.

Seismicity

The greater San Francisco Bay Area is recognized by Geologists and Seismologists as one of the most active seismic regions in the United States. Three major fault zones pass through the Bay Area in a northwest direction which have produced approximately 12 earthquakes per century strong enough to cause structural damage. The faults causing such earthquakes are part of the San Andreas Fault System, a major rift in the earth's crust that extends for at least 700 miles along western California. The San Andreas Fault System includes the San Andreas, Hayward, Calaveras Fault Zones, and other faults.

During 1990, the U.S. Geological Survey cited a 67 percent probability that a Richter magnitude 7 earthquake, similar to the 1989 Loma Prieta Earthquake, would occur on one of the active faults in the San Francisco Bay Region in the following 30 years. Recently, this probability was increased to 70 percent, as a result of studies in the vicinity of the Hayward Fault. A 23 percent probability is still attributed specifically to the potential for a magnitude 7 earthquake to occur along the San Andreas fault by the year 2020.

Ground Rupture - The lack of mapped active fault traces through the site, suggests that the potential for primary rupture due to fault offset on the property is low.

Ground Shaking - The subject site is likely to be subject to very strong to violent ground shaking during its life span due to a major earthquake on one of the above-listed faults. Current building code design should be followed by the structural engineer to minimize damages due to seismic shaking. The site should be considered to have a UBC Soil Type SE in the areas underlain by the old swale, and SD over most of the southeastern two-thirds of the site. Improvements should be designed to resist shaking from a Seismic Source Type A, located about 4 km from the site. Alternatively, site-specific accelerations may be utilized by the structural engineer for the design of the proposed improvements. The following accelerations were obtained by utilizing the EQFAULT computer program by T.F. Blake. The program provides a deterministic prediction of horizontal ground accelerations from more than 100 digitized faults. Then utilizing an attenuation relationship by Campbell and Bozorgnia (1994) for alluvial sites, a maximum-credible site acceleration of 0.54 g, and a maximum-probable site acceleration of 0.44 g were predicted for the property from a Maximum Credible Event of 7.5 RM, and Maximum Probable Event of 6.5 RM, on the adjacent Hayward fault. We note that the repeatable accelerations typically used for seismic design are generally considered to be on the order of 67% of the aforementioned peak values.

Landsliding - The subject site pad is generally level, and the sloping portions of the site are underlain by very stiff to hard colluvial materials. It is our opinion that the hazard due to seismically-induced, deep-seated landsliding is very low for the site. However, it is possible that some shallow sloughing of loose topsoils could occur on the steep rear slopes, however this should not affect the proposed building or lots atop the slope.

Liquefaction - Liquefaction most commonly occurs during earthquake shaking in loose fine sands and silty sands associated with a high ground water table. Based upon the subsurface investigation, the proposed building site is underlain by resistant clay-rich materials. Therefore, it is our opinion that liquefaction is unlikely to affect the subject property.

Ground Subsidence - Ground subsidence may occur when poorly consolidated soils densify as a result of earthquake shaking. Since the proposed building site is underlain at shallow depths by clay-rich resistant materials, the hazard due to ground subsidence is, in our opinion, considered to be low. However, the existing loose upper fill sands directly under the floor slab of the Cox Building may be subject to densification and settlements in the event of an earthquake (they will not liquefy as they are generally above the ground water table). The new building foundations will not be affected by settlements of these near surface sands, as the foundations will be embedded below the loose sands. However, the existing, or new, slabs-on-grade may experience significant settlements.

Lateral Spreading - Lateral spreading may occur when a weak layer of material, such as a sensitive silt or clay, loses its shear strength as a result of earthquake shaking. Overlying blocks of competent material may be translated laterally towards a free face. Such conditions were not encountered on the proposed building site, therefore, the hazard due to lateral spreading is, in our opinion, considered very low.

Temporary Support

A 20 foot tall retaining wall currently supports portions of the rear slope. Some of this wall will be located outside of the proposed new Avalon Bay building. This section of wall may remain in place. However, the wall currently derives some of its support from concrete framing in the adjacent building which is to be demolished. We recommend that tie-backs and walers be installed through the face of this wall to augment the temporary lateral support of the wall.

Where portions of the existing concrete walls need to be removed to permit construction of the new building walls, the slopes will need to be temporarily shored to provide a safe working environment. We suggest that in these areas, the shoring consist of cast-in-place drilled concrete piers with tie-backs to augment the lateral support.

Temporary shoring should be designed for a temporary active pressure of 35 pcf EFW acting from the ground surface to the base of the wall for a cantilevered wall design where some lateral displacement of the top of the wall can occur. Where the wall will be restrained by tie-backs, the active pressure should be increased by a uniform, rectangular pressure distribution of 8H, where H is the height of the wall from the tie-backs down to the pad subgrade elevation. Lagging may be required in the upper 3 to 5 feet of the shoring should significant amounts of topsoil or existing slide debris be present. (It is recommended that the existing slide debris on the eastern slope be removed prior to cutting into the hillside.)

To resist the active pressures a uniform, passive resistance of 500 psf may be assumed to act over 1.5 pier diameters below the elevation of the building pad subgrade. This lateral pressure may be augmented by lateral support from the tie-backs. Tie-backs should assume a non-bond length within a zone extending from the face of the wall back to a "plane of anchorage" defined by a 30 degree line (as measured from vertical) extending up into the hillside from the intersection of the cut pad grade and a point 5 feet into the hill behind the face of the piers.

The tie-backs and concrete piers should be designed by the shoring contractor, and the design should be reviewed by our office.

Site Preparation and Grading

Dewatering - The ground water elevations at the site range from about 5 feet deep to at the ground surface (or above), with the more shallow water elevations located towards the eastern side of the site. Dewatering of the site for construction purposes will likely be required. The dewatering program should be designed by a competent contractor experienced in such matters. (We note that free ground water was typically encountered between 6 and 12 feet below grade in the borings, and rose over time.)

Demolition - All debris resulting from the demolition of existing improvements should be removed from the site and may not be used as fill. Any existing underground utility lines to be abandoned, should be removed from within the proposed building envelope and their ends capped outside of the building envelope.

Due to the high elevation of the ground water table, and presence of soft soils under a thin veneer of stiffer fills, excavation of the building pad to a point three feet below current grade is likely to experience difficulty in heavy construction vehicle access. It is likely that excavations in some portions of the site will need to be made using excavators which reach into remove materials while working from a more stable pad. Stable pad and temporary roadway construction may also be required. Such construction may be possible by cement treating the soils, or reinforcement geotextile and gravel placement. Dewatering may assist in providing better construction access through the project.

Temporary grading should include measures to direct free water which may seep out of the ground, to flow to collection points for disposal.

Fill Placement - The placement of fills at the site is expected to include: utility trench backfill, retaining wall backfill, slab subgrade materials, slope reconstruction (see below) and finished drainage and landscaping grading. These and all other fills should be placed in conformance with the following guidelines:

Any vegetation and organically contaminated soils should be cleared from the building area, and from the portions of the rear slope to be cleared of slide debris, or to receive new fills.. All holes resulting from removal of tree stumps and roots, or other buried objects, should be over-excavated into firm materials and then backfilled and compacted with native materials.

Fills may use organic-free soils available at the site or import materials. Import soils should be free of construction debris or other deleterious materials and be non-expansive. *A minimum of 3 days prior to the placement of any fill, our office should be supplied with a 30 pound sample (approximately a full 5 gallon bucket) of any soil or baserock to be used as fill (including native and import materials) for testing and approval.*

All areas to receive fills should be stripped of organics, and loose or soft near-surface fill soils. New fills should be placed in lifts no greater than 6 inches thick (loose) and be compacted to at least 90 percent of their Maximum Dry Density (MDD) as determined by ASTM D-1557. If native expansive soils are used for structural fill at the site, then the soils should be placed at 3 to 5% over Optimum Moisture Content and be compacted to 90 percent of their MDD. Expansive soils should not be compacted under concrete slabs-on-grade or other lightly loaded improvements, as post-construction heave will cause movements of these materials. Native colluvial soils are likely to be too wet to compact to the required densities without moisture adjustments (i.e. drying).

Permanent cut and/or fill slopes should be no steeper than 2:1 (H:V). However, even at this gradient, minor sloughing of slopes may still occur in the future. Positive drainage improvements (e.g. drainage swales, catch basins, etc.) should be provided to prevent water from flowing over the tops of cut and/or fill slopes.

Slope Re-Grading - The overly steep slope on the northern side of Lot 12 Vernon Terrace should be regraded to create a slope which has gradients no steeper than 2:1. This can be accomplished by placement of soils generated by the cutting for the new wall, as engineered fills on the slope above the wall location. The grading may be conducted prior to wall construction and back cut, or the fill may be placed after the wall has been constructed.

All new fills to be placed on slopes steeper than 6 to 1 (horizontal to vertical) will need to be benched into competent native materials. The entire base of all benches should extend into competent colluvial soils, as identified in the field by representatives from our office. It should be anticipated that the outer edge of bench excavations will extend at least 2 feet below native grade. Keyways and benches should be sloped back into the hillside at a minimum 2% gradient.

For fills over 5 feet thick, or where deemed necessary by our personnel, a chimney drains should be provided at the back of any benches identified by our office in the field. Chimney drains should consist of a minimum 6 inch wide column of drain rock, wrapped with filter fabric, for at least half the height and for the full width of the bench. These systems should drain to 4 inch diameter perforated pipes, placed at the base of the drain rock. The pipes should consist of Schedule 40 PVC or SDR 35. No flexible, corrugated pipe may be used within any drainage system installed as part of this project. The bench drain pipes may connect to the back of wall drain pipes at the base of the slope. Alternatively, a solid line should be used to convey the water to an appropriate discharge point.

Pavement Subgrades - In pavement (concrete or asphalt) areas to receive vehicular traffic, any baserock materials should be compacted to at least 95 percent of their MDD. Also, the upper 6 inches of native soil subgrade beneath any pavements should be compacted to at least 90 percent of its MDD, with fill or import materials achieving 95 percent compaction. If drain rock is used for pavement support, no formal compaction is required, but the drain rock should be angular, and durable.

Temporary Excavations - Temporary, dry-weather, vertical excavations in clay soils on the building pad for the Avalon Bay Building should remain stable for short periods of time to heights of 5 feet **where not affected by the water table**. Deeper cuts may experience raveling and sloughing, particularly where below the water table and are likely to require shoring (see above). Excavations in the sandy fills under the Cox Building are unlikely to stand vertically over 3 feet.

All excavations should be sloped or shored in accordance with OSHA standards and with the previously provided recommendations.

Foundations

No single foundation type can effectively, and economically, address the variable conditions encountered at the site. The choice of foundations for use at the site for both structures is further restricted by the sensitivity of the existing Cox Building (block and plaster construction). Therefore, we have provided location specific recommendations for different foundation types (and often, alternatives) to support the various different loads and soil conditions to be encountered at the site.

Cox Building - Continuous Strip Footings - If continuous footings will be used to support the column loads for the EQE seismic retrofit work occuring along the Bay Place and Harrison Street sides of the building, then these footings should be designed and constructed to the recommendations presented below. The contractor should be advised that in excavating to the required depths, there may be problems with existing thick sections of concrete and brick footings, loose sandy soils, and shallow ground water. Measures to contend with these conditions should be identified by the contractor prior to the start of construction, and access to these mitigation measures readily available on short notice (if not pre-installed).

The footings should have a minimum depth of 5 feet below existing slab grade, and a minimum width which permits quality construction (we would suggest a minimum width of 24 inches). At this depth, the footings should have a long term bearing pressure on the soils which does not exceed 1000 psf. In calculating the bearing pressure, the weight of the embedded portion of the foundation may be neglected. This low bearing pressure is necessary to limit post-construction consolidation of the underlying soft soils to less than 1 inch, with differential settlements along the length of the footing less than 0.5 inches.

For seismic loading (downward only), the footings should be designed to limit the bearing loads to no more than 2500 psf. This significantly higher value is available, as this loading is not likely to induce vertical settlements of the underlying soft soils. Lateral loads, if any, may be resisted by base friction (coefficient of friction of 0.35), and by passive resistance against the face of the footing, assuming a uniform value of 250 psf.

Cox Building - Helical Augers and Grade Beams (Alt.) - Alternatively, the new continuous footings along Bay Place and Harrison Street may consist of shallow grade beams (footings) further supported by deeper Chance Augers (Helical piers). This may make the construction easier, but will require the retention of a specialty contractor to design and install the helical piers. This foundation system will be primarily designed by the contractor's in-house engineering personnel, however, the design should accommodate the following requirements:

The grade beams should have a minimum depth of 24 inches below existing slab grade in order to expose any potential existing foundations or footings buried in the loose sands under the existing slabs. The new piers will need to support both the loads from the columns, plus the load of the grade beam itself.

The new helical piers will need to penetrate a minimum of 15 feet along the Harrison Street side footings, and a minimum of 20 feet along the Bay Place side footings. Actual embedment depth will need to be calculated by the designer for this system based upon the information contained in our borings ***(Note: the blow counts on our boring logs should be converted using a factor of 0.6 to convert the Mod. Cal. Sampler driving energy to equivalent SPT values.)***. The vertical settlements of the footing should be kept to less than 1 inch.

Lateral resistance (if required) will need to be developed on the face of the grade beam, or through inclined anchors. Passive resistance on the face of the grade beam should be assumed using a uniform pressure distribution of 250 psf.

There may be areas where the auger holes need to be pre-excavated or cored to penetrate through thick sections of existing brick and concrete materials (as encountered by Borings GF-8 and exposed in the EQE test pit). The contractor should be prepared for this potential.

Cox Building - Isolated Back Wall Foundations - The foundations along the rear block wall of the Cox building should be structurally incorporated into the new foundations for the Avalon Bay project building (see below). The "Cox" and "Avalon Bay" structural engineers will need to coordinate their efforts to make sure that: 1) post construction settlements do not exceed 1 inch along this common foundation; 2) seismic loads are adequately transferred through that footing; and, 3) allowable bearing capacities are not exceeded.

Avalon Bay Foundations - Stiff Soils - In the southeastern two-thirds of the site, the foundations for the new Avalon Bay complex may consist of conventional spread footings on the very stiff sandy clay soils. The foundations should be designed by the structural engineer based upon the following parameters:

The footings should be designed to exert pressures on the ground which do not exceed 3000 psf for Dead plus Long-Term Live Loads. The weight of the embedded portion of the footings may be neglected when determining bearing pressures. Lateral pressures may be resisted by friction between the base of the footings and the ground surface. A friction coefficient of 0.35 may be assumed. These values may be increased ⅓ for transient loads (i.e. seismic and wind).

Footings should be embedded a minimum of 3 feet below pad grade. All footings should bear on very stiff soils, as determined by our office in the field. Localized deepening of footings may be required if variable conditions are encountered during construction.

Footings should be founded below an imaginary line projecting at a 1:1 slope from the base any adjacent, parallel utility trenches.

If the above recommendations are followed, total foundation settlements should be less than 1.5 inches, while differential settlements between similarly loaded columns should be less than ¾ inches.

Avalon Bay Foundations - Soft Soils - In the western corner of site, an old buried swale is filled with soft/loose soils. In order to achieve a foundation system which will not experience excessive settlements over these soils, the Avalon Bay structural engineer will either need to construct a "waffle" style mat of interlocking grade beams (to limit near surface loads), or to transfer structural loads to the more competent deeper soil materials. We have recommended the use of Cement-Treated Soil Columns (CTS Columns) to achieve this deeper penetration, without the need for ground vibrations; casing of open holes; or removal of potentially contaminated soils. As an alternative, it would also be acceptable to our office if Helical Augers were used to support the loads (see discussion above for the Cox Building foundations).

The foundations for the new Avalon Bay complex may be designed as conventional spread footings which will rest atop a series of cement treated soil columns (Geo-Columns, Auger piles). These columns will be designed by the specialty contractors who install these applications, but should generally include the following design guidelines:

These cement treated soil (CTS) columns should be spaced about 2 diameters on center in order to help distribute loads to the deeper, more stable soils, from the overlying surface "pier cap/ footing". The CTS columns should span from corner to corner under the entire footprint of the "footings". The "footings" should be designed by the structural engineer based upon an assumed "surface bearing capacity". Proximate to the Cox Building, the surface footing should cover an area to provide an equivalent bearing capacity of 1500 psf for the Dead plus Long-Term Live Load. Seismic loadings on these footings may have an increased effective bearing pressure of 2500 psf, as this higher pressure will not contribute to settlements of the soft underlying soils. Settlements of these foundations should be less than ¾ inch, with differential settlements between similarly supported (and loaded) foundations less than ½ inch.

Away from the Cox Building, footings may be designed for an increased "equivalent surface bearing capacity", provided the increased settlements can be tolerated by the building, and provided the increased pressures do not impact the Cox Building. For an isolated footing to avoid impacting the Cox Building, the edge of the footing must be located a minimum of one footing width away from the edge of the Cox Building (for example, the edge of a 10 by 10 foot wide footing should be located at least 10 feet from the Cox Building to be a candidate for increased bearing pressure). Strip footings should be located a minimum of 1.25 times the footing width away from the Cox Building.

If the foundations will not impact the Cox Building, and additional differential settlements can be tolerated, then the bearing pressures may be increased. These "footings" may be sized for an "equivalent bearing pressure" of 3000 psf for Dead plus Long Term Live Loads (and 4000 psf for seismic). Under these pressures, total settlements should be less than 1.5 inches, with differentials between similarly supported and loaded columns of about 1 inch.

Minimum CTS column depths have been plotted on Figure 5 attached to this report. The specified minimum CTS column lengths vary in 5 foot increments from 10 to 20 feet deep below pad grade (<u>not</u> base of footing). In no case should the CTS columns be less than 10 feet deep.

Avalon Bay Foundations - Soft Soils (Alt.) - As an alternative to CTS columns, those portions of the building where soft soils exist may be constructed as a waffle-style foundation system. This system would use interlocking concrete grade beams to distribute the concentrated column loads over the entire building area footprint. The anticipated bearing pressures should not exceed 1000 psf (not including the weight of the "submerged" waffle grade beams. At this pressure, settlements in the center of the slab should not exceed 1.5 inches, with settlements along the Cox Building expected to be less than ¾ inch along most of the common wall, and less than ½ inch at the corner towards Harrison Street.

The waffle-style mat should be designed over the soft site soils assuming a Modulus of Subgrade Reaction on the order of 20 tcf (k_1 for a 1 by 1 foot square plate). This will likely result in grade beams which are a minimum of 5 feet deep, and spaced no further apart than about 15 to 20 feet.

Due to the potential for problems with construction traffic access, soft soils, and shallow ground water conditions, the construction of several deep trenches may be difficult and uneconomical. This should be evaluated by your project construction consultant.

Retaining Walls

The proposed retaining walls at the site will all be located in the southeastern portions of the project site where the stiff clay soils are present. Therefore, these walls may be constructed with conventional spread footing (heel and toe) foundations. These foundations may be further augmented with tie-backs if desired or by transferring the loads laterally through the floor diaphragm for the parking garage (due to the anticipated high lateral loads).

Active Wall Forces - Any unrestrained retaining walls required for the proposed construction should be designed to resist an active pressure of 40 pcf Equivalent Fluid Weight (EFW) in supporting soils with retained slopes less than 4:1 (H:V). An active pressure of 55 pcf EFW should be utilized for retained slopes with an inclination of 2:1 (H:V). Where retained slopes are greater than 4:1, though less than 2:1, the designer should linearly interpolate between 40 and 55 pcf EFW.

Any restrained retaining walls required should be designed for the aforementioned active pressures with an additional uniform pressure of 8H psf, where H is the height of the wall in feet. We leave it to the design professional's judgement in determining whether a wall is restrained or not.

All retaining walls should also be designed to resist a point load applied at the midpoint of the wall, equal to ½ the maximum applied surcharge (if any).

Lateral Resistance Pressures - The active pressures may be resisted using either friction on the base of the wall footing (assuming a coefficient of friction of 0.4), or passive resistance (400 pcf EFW) against a shear key. Our office should be contacted for allowable resistance values, as the location of the shear key will affect the available passive resistance available. It is unlikely that both passive and frictional resistances will be permitted to act together. The above values may be increased ⅓ for transient loads.

Tie-Backs - If required, tie-backs can be used to augment the support of the retaining wall system (thereby limiting bending moments in the stem and foundation overturning pressures). Permanent tie-backs may be designed using the same principals as used for temporary shoring tie-back design, but factors of safety between temporary and permanent should be appropriately increased. Tie-backs may consist of grouted anchors (e.g. Ebo-rods), or mechanical anchors (e.g. Chance Augers).

Bearing Pressures - The foundations for the new wall may consist of conventional spread footings. These footings should be designed using an allowable maximum bearing pressure of 4000 psf, at a minimum depth of 5 feet below existing grade, and 3 feet below finished grade. All footings must be supported on competent materials as approved by our office in the field. Average vertical loads on the footings should not exceed 3000 psf to limit potential differential settlements with adjacent foundation systems.

Back-of-Wall Drainage - The above values have been provided assuming that back-of-wall drains will be installed to prevent build-up of hydrostatic pressures behind all walls. This drainage system may consist of a prefabricated drainage panel (i.e. Miradrain) or a gravel and filter fabric type system. We also recommend that any interior retaining walls, or walls through which efflorescence transmission would be undesirable, should be waterproofed. The waterproofing should be specified by the designer. Additionally, the ground surface above all walls should form a drainage swale to carry water to the sides of the wall. Excess surface water should not overtop the retaining wall.

The back-of-wall drain systems should be installed with a minimum 6 inch diameter perforated pipe placed a minimum of 12 inches below the top of the interior floor slab. Perforations should be placed face-down (at 5 and 7 o'clock). The perforated pipe should connect to a solid discharge line, which will discharge to the street, or other approved location away from the new structure. This solid line should not connect directly to surface water drain lines (i.e. downspout and area drain lines).

If used, the gravel system should consist of a minimum 12 inch wide column of drain rock (⅜ to ¾ inch clean, crushed rock) extending the full width of the wall. The rock should continue to within 12 inches of finish grade. Prior to backfilling with the drain rock, a layer of filter fabric (Mirafi 140N or approved equivalent) should be placed against all soil surfaces to separate the rock and soil. The filter fabric should wrap over the top of the gravel and then a 12 inch thick cap of native soils should be placed at the top of the drain. If concrete flatwork is to directly overlay the back-of-wall drain then the soil cap should be eliminated.

If prefabricated drainage panels are used, a packet of filter fabric-wrapped drain rock should be placed around the perforated collector pipe at the base of the panel. The tops of the panels should be sealed and secured in accordance with the manufacturer's recommendations.

We note that Caltrans Class II permeable rock may be utilized in lieu of clean drain rock and filter fabric. The Class II permeable rock needs to be compacted into place, and needs to be certified by the quarry or rockery that it meets the Caltrans Class II permeable rock specifications. Additionally, the perforated collector pipes will need to be wrapped in a filter fabric sock to prevent the permeable rock from washing into the pipe.

Slabs-on-Grade

The lower story parking structure, and exterior landscaping patios or walkways may have concrete slab-on-grade floors.

Slab Design - To help reduce cracking, we recommend garage slabs be a minimum of 5 inches thick and be nominally reinforced with #4 bars at 24 inches on center, each way. Exterior landscaping slabs may be 4 inches thick. Slabs which are thinner or more lightly reinforced may experience undesirable cosmetic cracking. However, actual reinforcement and thickness should be determined by the structural engineer based upon anticipated usage and loading.

In large slabs (e.g. patios, garage, etc.), score joints should be placed at a maximum of 15 feet on center. In sidewalks, score joints should be placed at a maximum of 5 feet on center. All slabs should be separated from adjacent improvements (e.g. footings, columns, etc.) with expansion joints.

Due to the highly expansive nature of the site soils, the upper 18 inches of soil subgrade under the garage slabs should either be lime treated, or an additional 6 inches should be removed and replaced with drain rock (for a total section of 18 inches of drain rock, see Sub-Slab Drainage section below).It would be prudent (though not required) to underlay all landscaping slabs with at least 12 inches of non-expansive materials. This will help to reduce future expansive soil movements of the slabs. Slabs which are not underlain by this non-expansive material may undergo excessive seasonal shifting.

Interior slabs, and slabs through which moisture transmission is undesirable, should be underlain by 2 inches of sand over 4 inches of ¾ inch drain rock. The sand and drain rock should be separated by a vapor barrier (e.g. visqueen). The 6 inches of granular subgrade may be included as part of the 12 inches of non-expansive materials.

As stated previously, in pavement (concrete or asphalt) areas to receive vehicular traffic, all baserock materials and the upper 6 inches of import soils should be compacted to at least 95 percent of their MDD. Also, the upper 6 inches of native soil subgrade beneath any pavements should be compacted to at least 90 percent of its MDD. Drain rock sections over 2 feet in thickness should be vibrated into place using a "turtle" or vibratory plate.

To reduce post-construction expansive soil movements (i.e. heave) of any slabs, care should be taken to keep the subgrade moist for an extended period of time (two to three weeks) prior to pouring the slabs. *Shrinkage cracks should not be allowed to develop in the soil beneath any proposed slabs.*

Sub-Slab Drainage - due to the potentially high ground water conditions, recommend that the floors be underlain by a minimum of 12 inches of drain rock. At the base of the drain rock, at distances not to exceed 25 feet, perforated collector pipes should be installed to convey water which seeps into the drain rock layer out to an appropriate discharge facility. The need to treat any removed water should be determined by Levine-Fricke based upon their current investigation. It may be desirable to have the water from different areas collected in separate systems to minimize the potential amount of water requiring treatment. Where practical, the subslab drainage system should discharge into the storm sewers in the street.

Cox Building - Slab Treatment - The existing slabs in the northern third (portion towards Harrison Street) are out of level, and are locally spanning over a void up to 4 inches tall. The slab does not appear to be a structural improvement, and its "failed" condition is purely a cosmetic issue. Therefore, the owner has the option of addressing this condition in any one of several ways, including:

1 - Do nothing - the slab may simply be left in place in its current condition. However, the owner must accept the potential that some future movements, cracking, and distortions may occur in the future, if the slab decides to drop down back in contact with the underlying soils. This may occur as a result of earthquake shaking, additional loads due to storage of materials, or due to time alone. At that time, the slab may be replaced, or may be covered over as desired, but that would obviously be more expensive than if the same work were performed now, while the slab is accessible.

2 - Level - the application of a leveling course of cementaceous material may be poured on top of the slab to create a more level condition of the slab surface. This will not correct the voids which exist under the slab, so it is merely a cosmetic treatment of the surface condition, and the slab is subject to the same potential problems as discussed above under "Do Nothing".

3 - Grout - the void beneath the slab may be filled with grout so that contact with the sandy soils are achieved again and the slab is supported. Future settlements are possible, but should be very limited, and are unlikely to require significant repairs.

4 - Remove and Replace - the sections of the slabs which have settled, and/or contain voids beneath the slab may be removed and then replaced with new concrete slabs. The new slabs should have a minimum of 4 inches of drain rock under the slabs to limit the potential for moisture penetration through the slab, which can affect floor surface coverings. Over the gravel a moisture barrier (visqueen) and a 1 inch layer of sand may be placed to further limit moisture penetration if desired. The new slabs should have a minimum thickness of 4 inches and be reinforced with a minimum of #3 bars at 24 inch centers. Score joints should be placed at appropriate locations to limit cracking due to hard surface under the slabs, and due to shrinkage during curing. Alternatively, cracks should be patched after the slab has cured.

Drainage

Due to the expansive nature of the site soils, it will be important to provide good drainage improvements at the property.

Surface Drainage - Adjacent to any buildings, the ground surface should slope at least 4 percent away from the foundations within 5 feet of the perimeter, except where the perimeter foundation is a retaining wall. In these locations, the ground may slope towards the building with a surface swale provided to convey the water to a storm water drainage system. Impervious surfaces should have a minimum gradient of 2 percent away from the foundations.

Surface water should be directed away from all buildings into drainage swales, or into a surface drainage system (i.e. catch basins and a solid drain line). "Trapped" planting areas should not be created next to any building foundations without providing means for drainage.

All roof water should be conveyed via pipeline to the storm drain system, or should discharge onto paved surfaces which drain away from the structure. Roof water, and other surface drainage systems should not connect to the sub-slab drain, back-of-wall drain, or any other perforated subsurface drainage system.

Drainage Materials - Drain lines should consist of hard-walled pipes (e.g. Schedule 40 PVC or SDR 35). Corrugated, flexible pipes may not be used in any drain system installed at the property.

Surface drain lines (e.g. downspouts, area drains, etc.) should be laid with a minimum 2 percent gradient (¼ inch of fall per foot of pipe). Subsurface drain systems (e.g. footing drains) should be laid with a minimum 1 percent gradient (⅛ inch of fall per foot of pipe).

Utility Lines

All utility trenches should be backfilled with compacted native clay-rich materials within 5 feet of any buildings. This will help to prevent migration of surface water into trenches and then underneath the structures' perimeter. The rest of the trenches may be compacted with other native soils or clean imported fill. Only mechanical means of compaction of trench backfill will be allowed. Jetting of sands is not acceptable. Trench backfill should be compacted to at least 90 percent of its MDD. However, under pavements, concrete flatwork, and footings the upper 12 inches of trench backfill must be compacted to at least 95 percent of its MDD.

Plan Review and Construction Observations

The use of the recommendations contained within this report are contingent upon our being contracted to review the plans, and to observe geotechnically relevant aspects of the construction.

We should be provided with a full set of plans to review at the same time the plans are submitted to the building/planning department for review. A minimum of two working weeks should be provided for review of the plans.

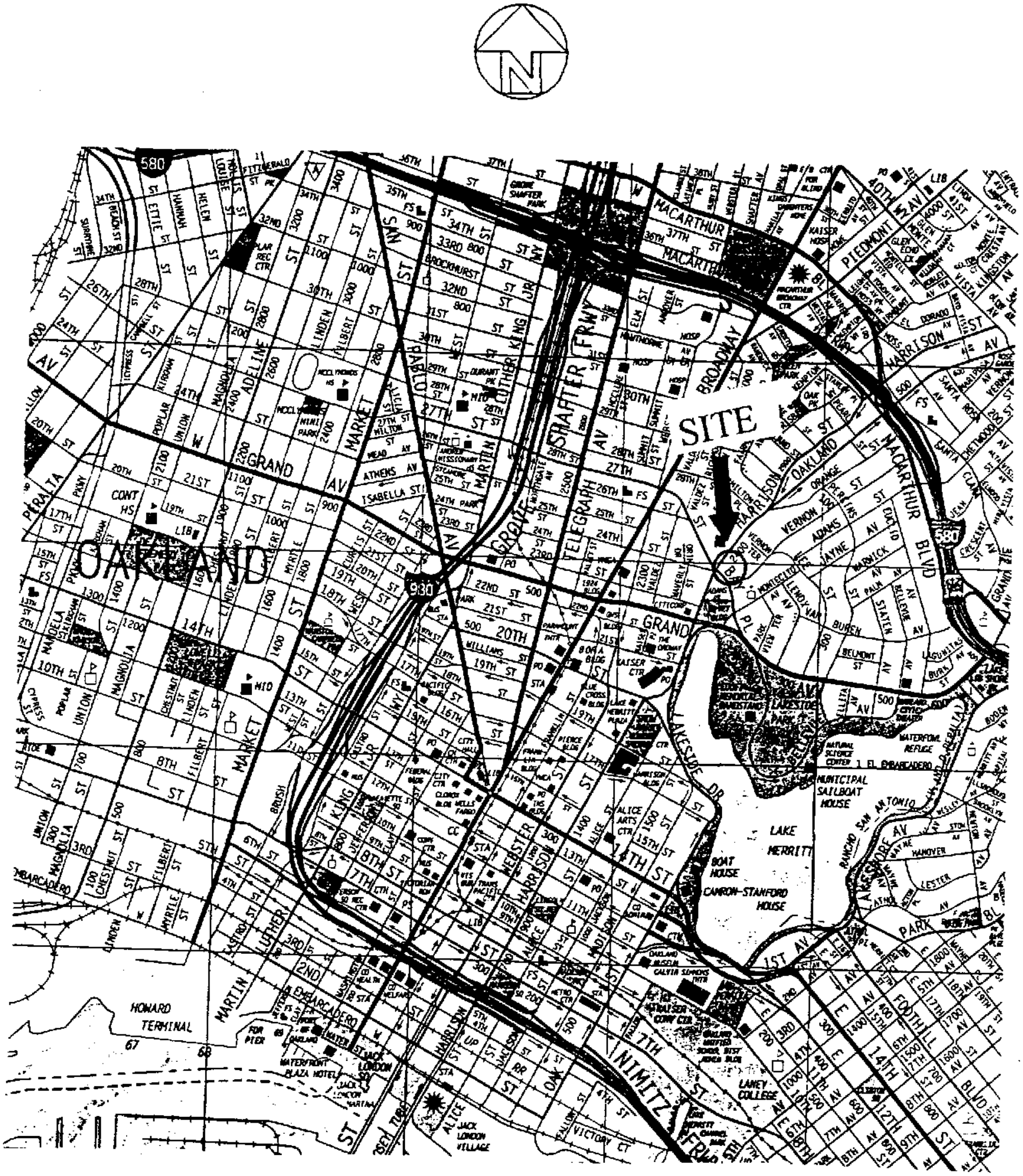
At a minimum, our observations should include: key and bench excavations; compaction testing of fills and subgrades; footing excavations; geo-column drilling; slab and driveway subgrade preparation; installation of any drainage system (e.g. back-of-wall, sub-slab, and surface), and final grading. A minimum of 48 hours notice should be provided for all construction observations.

LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers for aiding in the design and construction of the proposed development. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments and conclusions presented in this report were based upon information derived from our field investigation and laboratory testing, and the preliminary reports and investigations performed by Lowney Associates. Conditions between, or beyond, the borings and soundings may vary from those encountered. Such variations may result in changes to our recommendations and possibly variations in project costs. Should any additional information become available, or should there be changes in the proposed scope of work as outlined above, then we should be supplied with that information so as to make any necessary changes to our opinions and recommendations. Such changes may require additional investigation or analyses, and hence additional costs may be incurred.

Our work has been conducted in general conformance with the standard of care in the field of geotechnical engineering currently in practice in the San Francisco Bay Area for projects of this nature and magnitude. We make no other warranty either expressed or implied. By utilizing the design recommendations within this report, the addressee acknowledges and accepts the risks and limitations of development at the site, as outlined within the report.



Source: Thomas Bros. Guide

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Figure 1 - Site Location

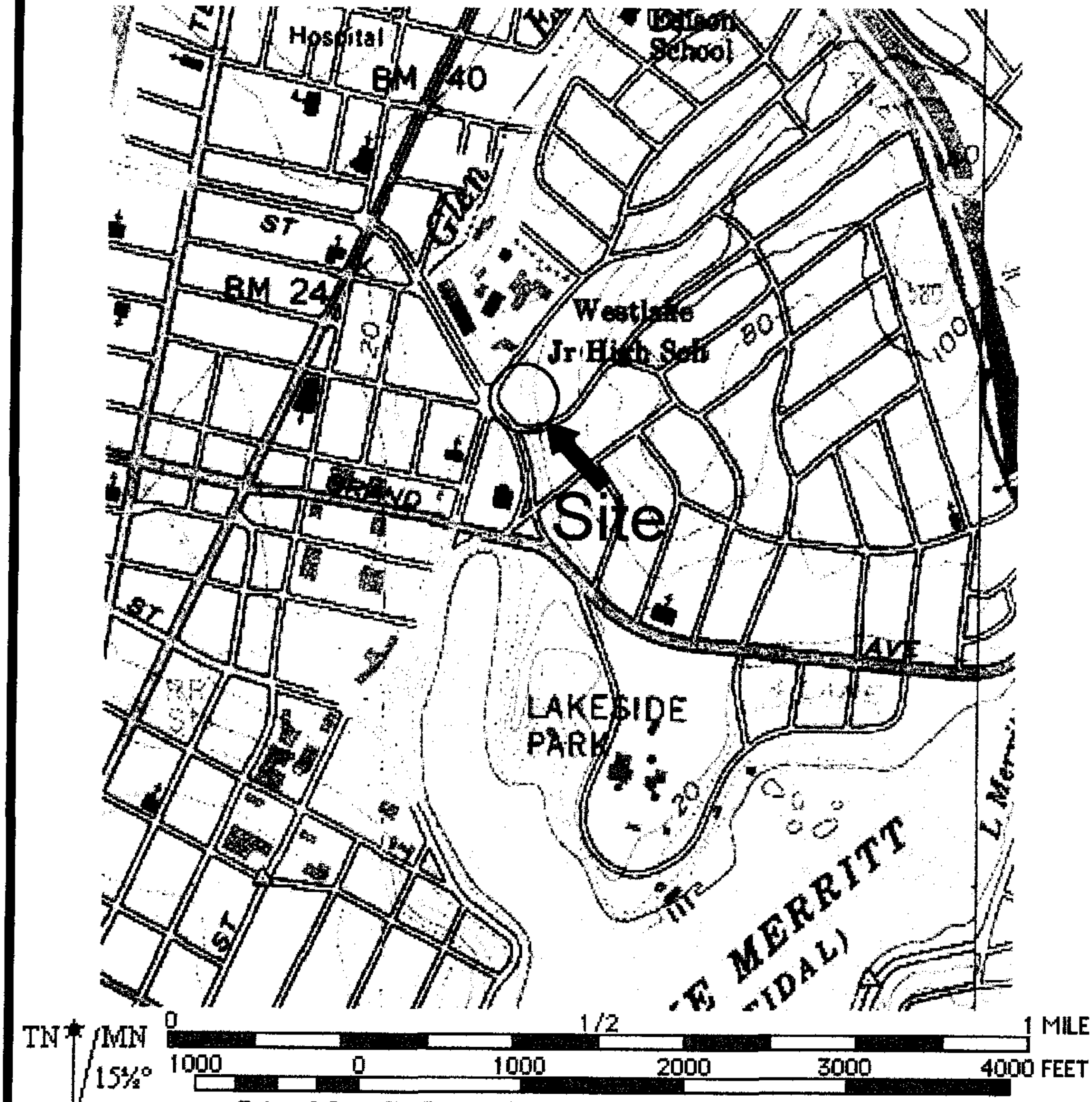

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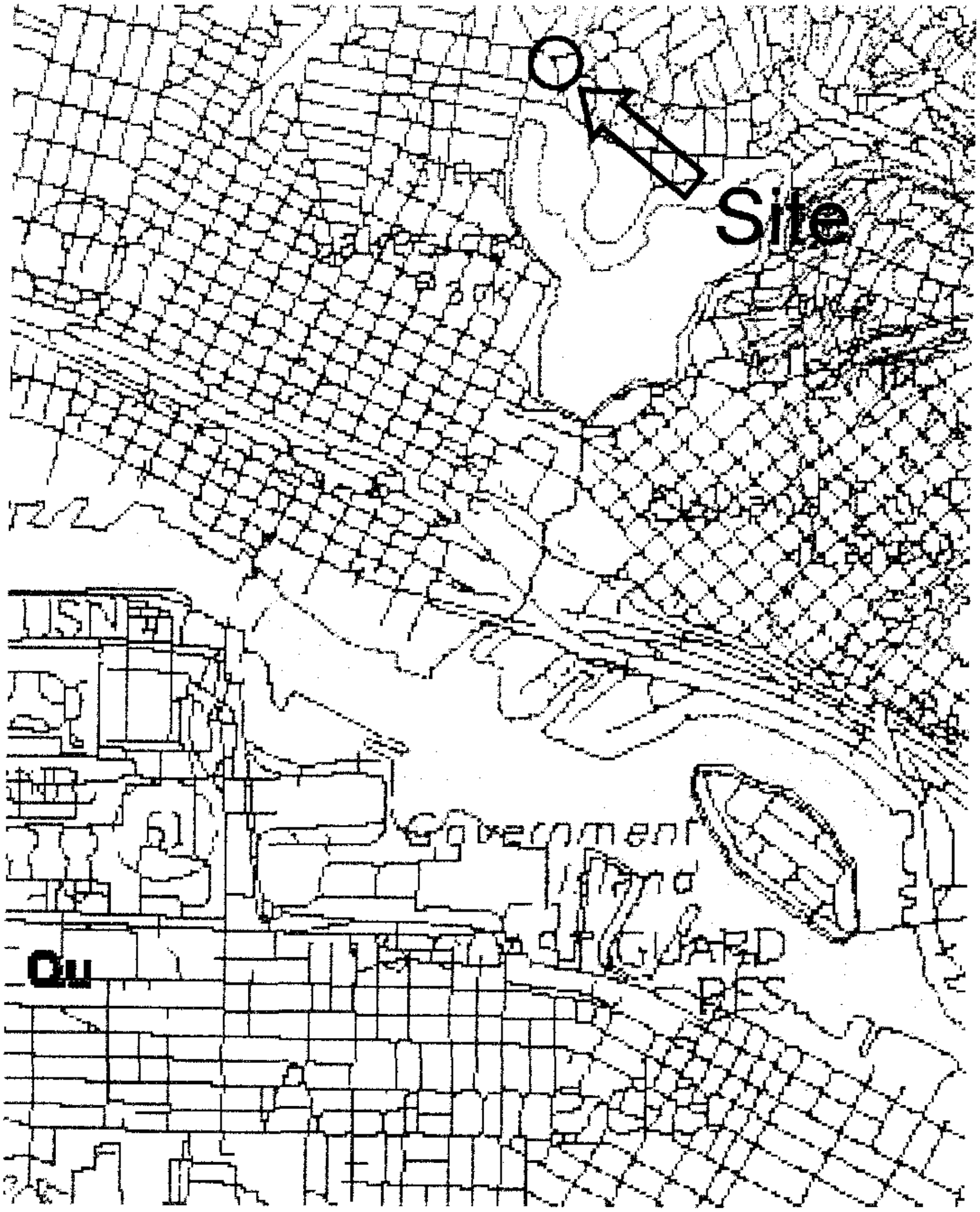
Figure 2 - Vicinity Topography



MAP EXPLANATION

☐ **Qu** **Undivided surficial deposits**

Source: *Preliminary Geologic Map Emphasizing Bedrock Formations in Alameda County, California: Derived from the Digital Database Open-file 96-252*, by Graymer R.W., et al, (1996).

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Figure 3 - Geologic Map

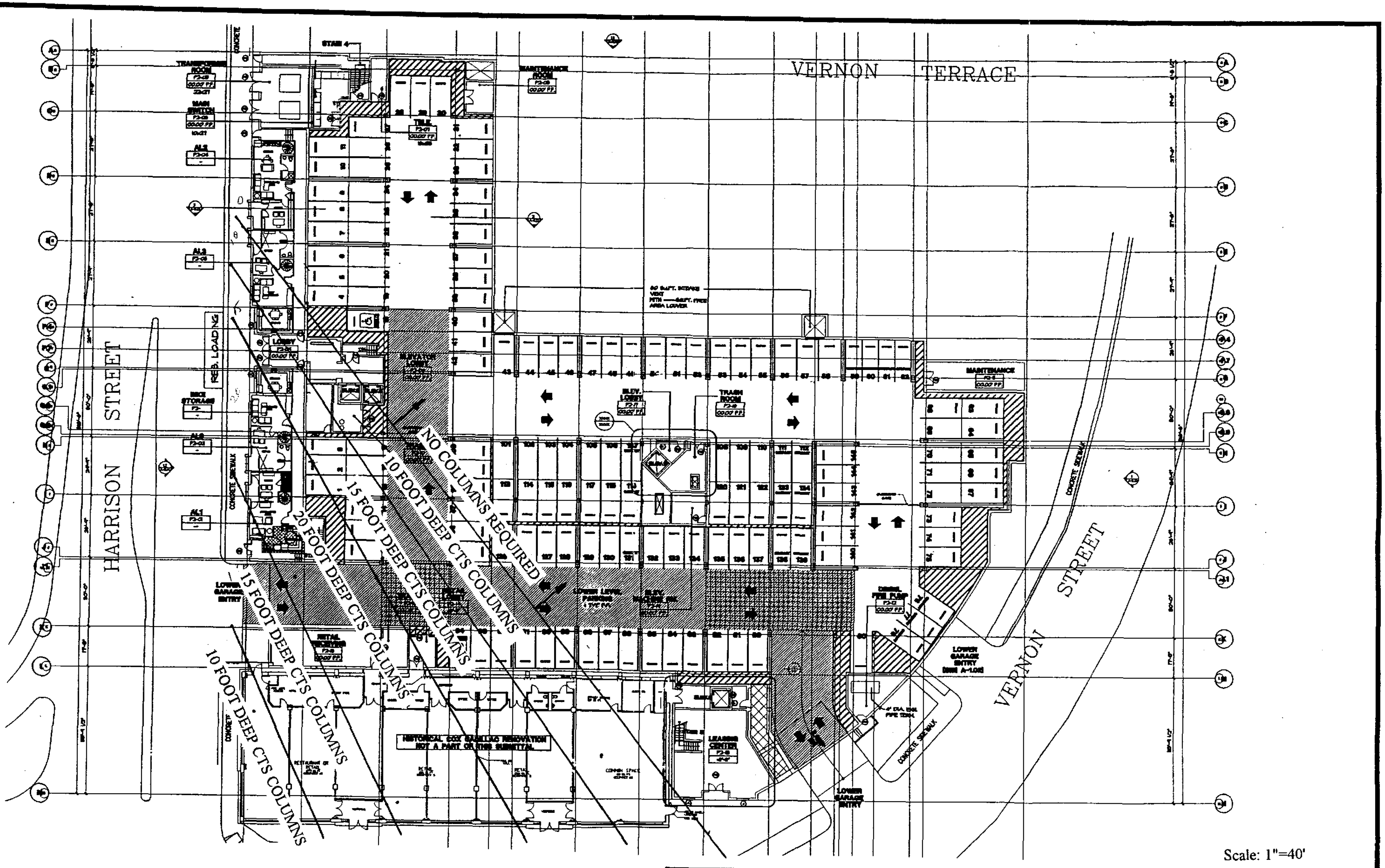

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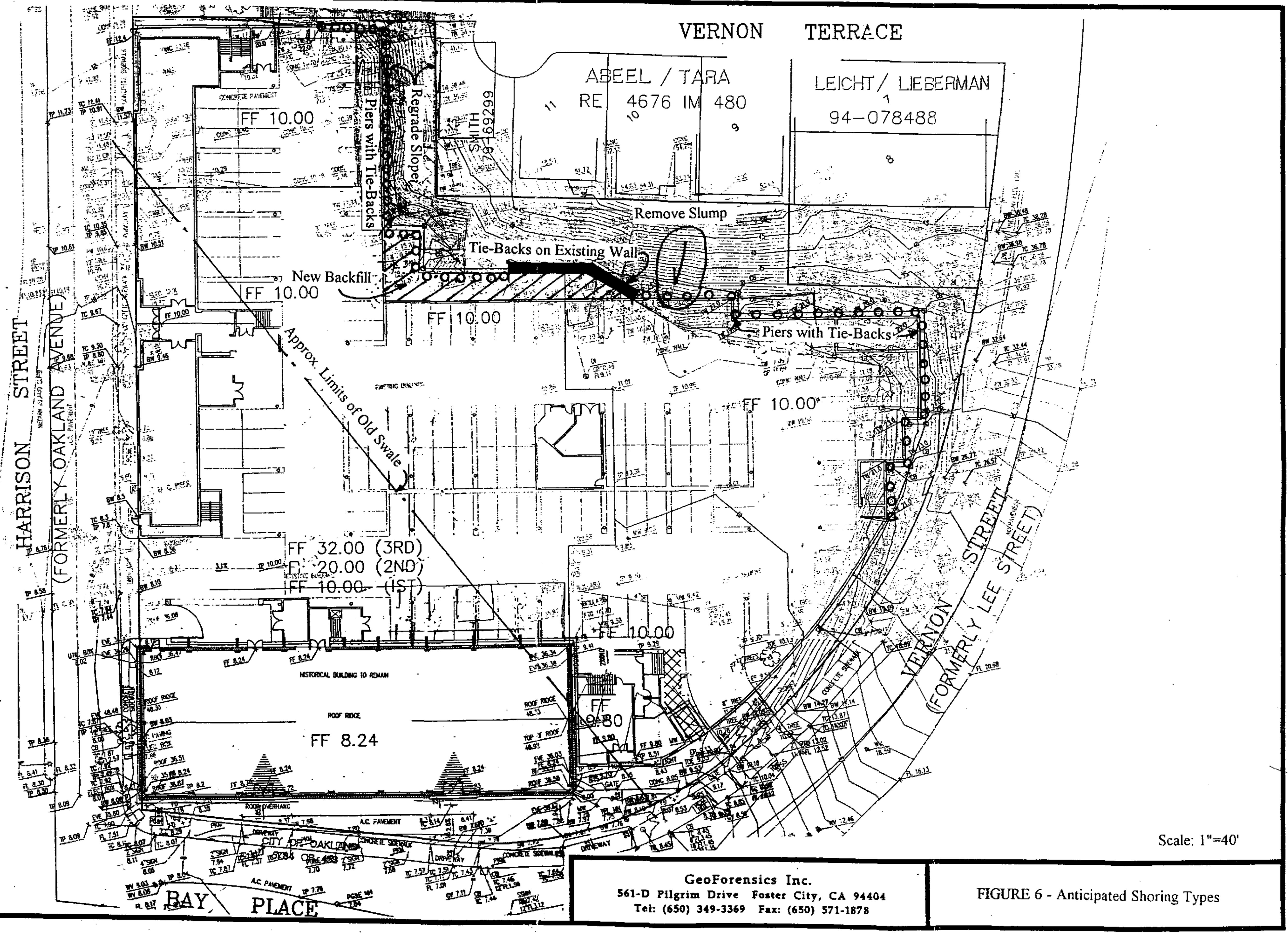
FIGURE 5 - Cement Treated Soil Column Depths



Scale: 1"=40'

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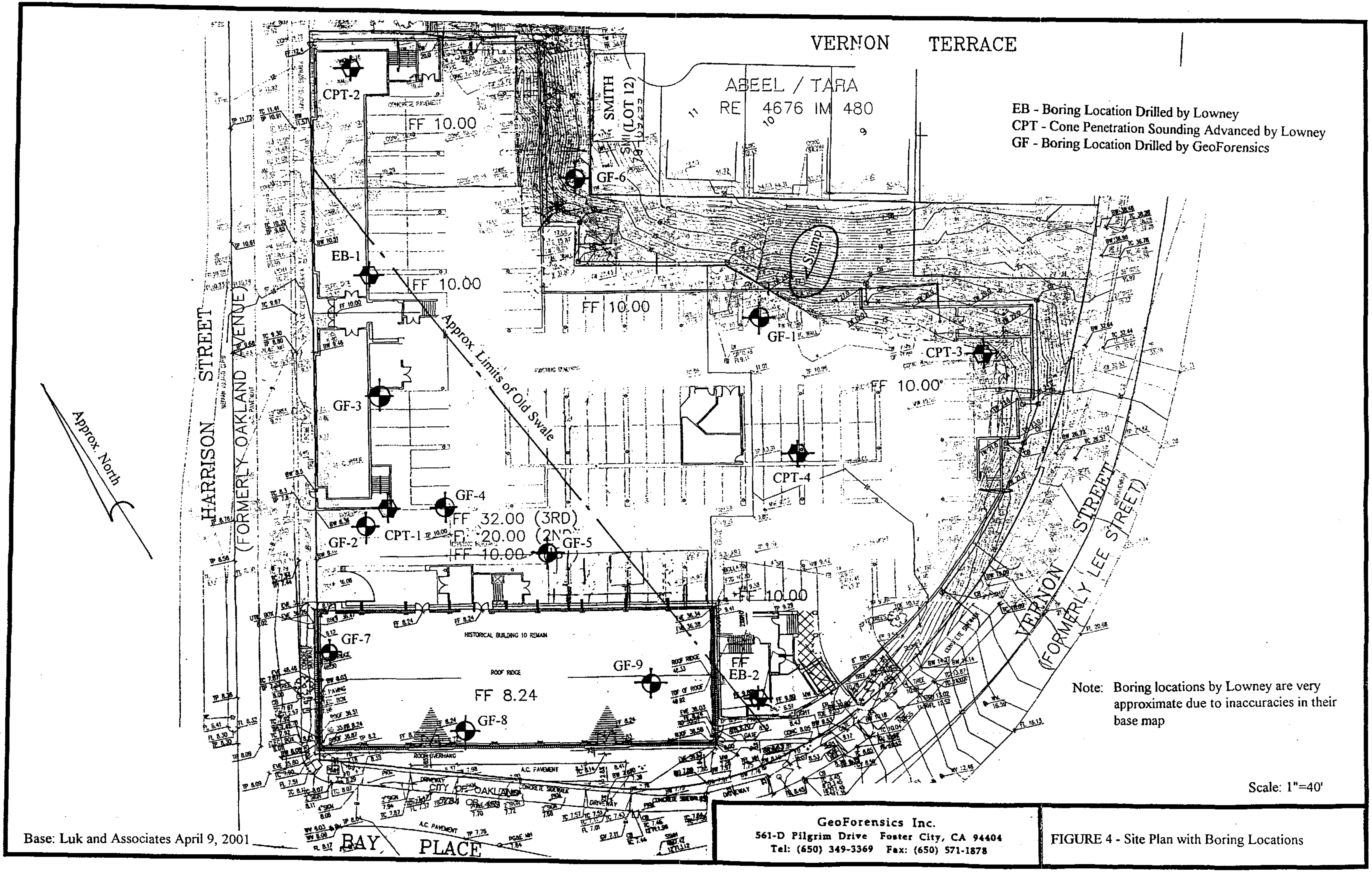
FIGURE 6 - Anticipated Shoring Types



EB - Boring Location Drilled by Lowney
CPT - Cone Penetration Sounding Advanced by Lowney
GF - Boring Location Drilled by GeoForensics

Note: Boring locations by Lowney are very approximate due to inaccuracies in their base map

Scale: 1"=40'

Base: Luk and Associates April 9, 2001

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FIGURE 4 - Site Plan with Boring Locations

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				silty CLAY with sand - mottled grey & orange-brown; slightly moist		
5				CLAY - green-grey; slightly moist; soft		
	2 - 1	◢	11		94.8	29.3
10				clayey SILT - black; organic; very moist to wet; soft	98.1	23.0
15	2 - 2	◢	21	sandy CLAY - dark grey & black; wet; stiff		
20	2 - 3	◢	28	sandy CLAY with trace fine gravel - mottled orange & grey-brown; slightly moist; very stiff	101.7	24.5
25						
30						
35						

Groundwater at 12 feet after 1 hour (not stabilized).
Bottom of boring at 21.5 feet
Drilled on 04/05/01
Logged by dd/ba
Mobile B-24 drilling rig
Modified California sampler
140# hammer

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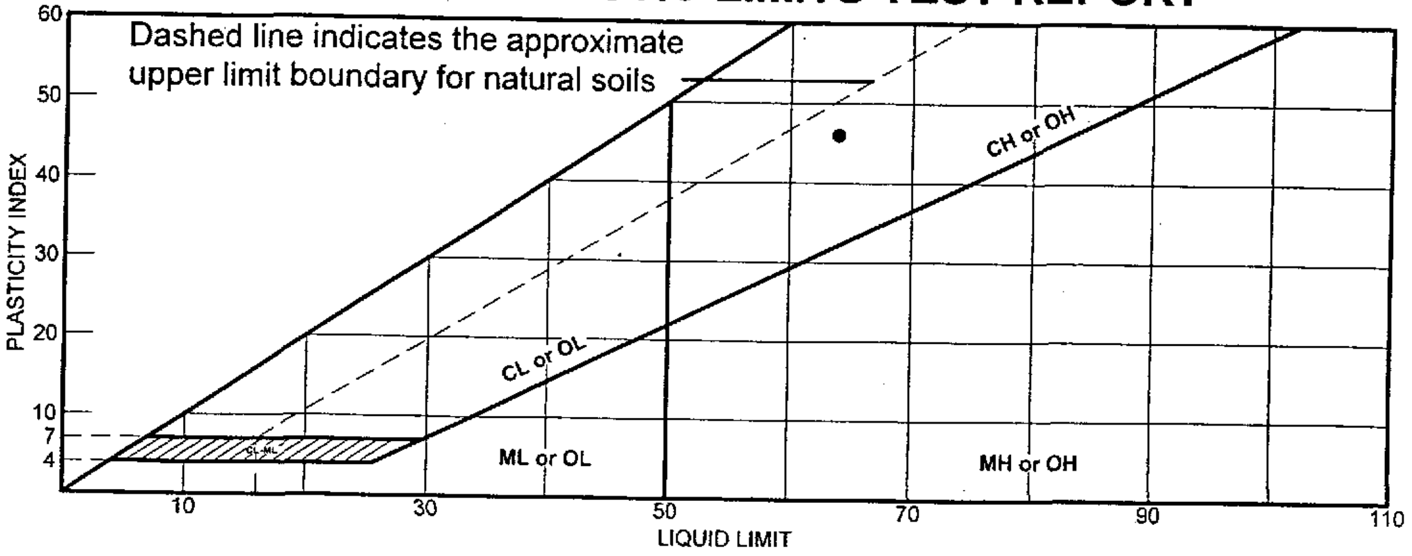
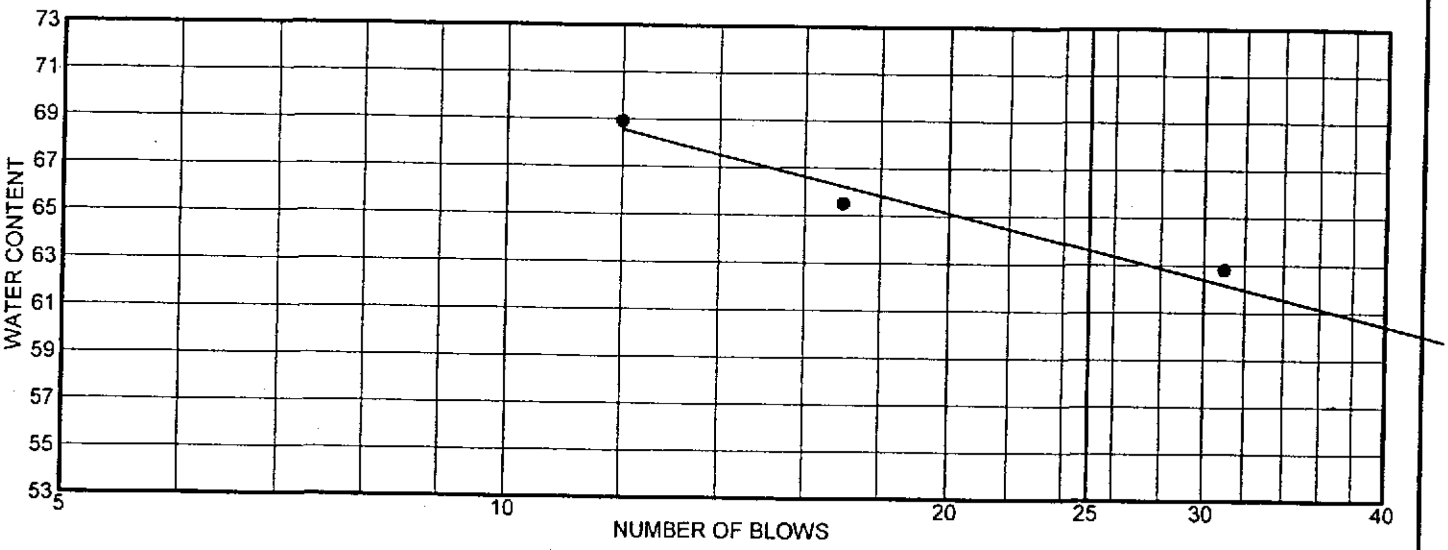
Figure A2 - Log of Boring GF-2

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	olive gray CLAY with sand	64	18	46			

Project No. 010-1192 **Client:** GeoForensics

Project: 201071 / Avalon

● **Source:** 2-2 **Elev./Depth:** 16

Remarks:

●

LIQUID AND PLASTIC LIMITS TEST REPORT

COOPER TESTING LABORATORY

Plate

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 Inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
5	3 - 1	◢	9	CLAY - dark grey-black; slightly moist; medium stiff	94.2	26.0
10	3 - 2	◢	21	sandy CLAY - dark grey-black; slightly moist; stiff	101.0	24.7
15	3 - 3	◢	27	clayey fine SAND/fine sandy CLAY - mottled orange & grey-brown; slightly moist; medium dense/very stiff		
20	3 - 4	◢	29	fine sandy CLAY with trace fine gravel - mottled orange & grey-brown; slightly moist; very stiff		
25						
30						
35						

Groundwater encountered at 13 feet, rose to 10.5 feet after 1 hour.
Bottom of boring at 20 feet
Drilled on 04/05/01
Logged by ba
Mobile B-24 drilling rig
Modified California sampler
140# hammer

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Figure A3 - Log of Boring GF-3

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				silty CLAY with trace sand & fine gravels - tan-brown; slightly moist; soft		
	4 - 1	◢	8		93.6	25.4
5				sandy CLAY - mottled orange and reddish dark brown; moist; soft	98.5	23.0
10	4 - 2	◢	11	CLAY - black; slightly moist; medium stiff	99.0	24.0
15	4 - 3	◢	25	sandy CLAY with few gravels - grey; slightly moist to moist; very stiff	118.6	15.8
				gravelly SAND - brown; wet; loose	108.0	19.0
20	4 - 4	◢	25			
				sandy CLAY with few fine gravels - mottled orange & grey-brown; slightly moist; very stiff		
25	4 - 5	◢	28	as above - stiff	109.1	18.0
30	4 - 6	◢	30	CLAY with fine sand & reeds? - mottled orange & grey-brown; slightly moist; very stiff		
35				Groundwater at 6.5 feet after 1 hour. Bottom of boring at 30 feet Drilled on 04/05/01; Logged by ba Mobile B-24 drilling rig Modified California sampler; 140# hammer		

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Figure A4 - Log of Boring GF-4

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
	5 - 1			4" CONCRETE SLAB sandy CLAY - mottled orange & brown; firm; moist CONCRETE 3 feet thick		
5				Practical refusal at 6 feet.		
10						
15						
20						
25				No groundwater encountered. Bottom of boring at 6 feet Drilled on 04/05/01 Logged by dd/ba Mobile B-24 drilling rig Modified California sampler 140# hammer		
30						
35						

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Figure A5 - Log of Boring GF-5

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				sandy CLAY/clayey SAND - dark brown; slightly moist; firm to stiff		
5						
	6 - 1	◢	42		94.9	14.6
				clayey gravelly SAND - orange-brown; slightly moist; medium dense		
10						
	6 - 2	◢	88	as above, but with lenses of silty fine SAND with trace clay - medium dense to dense	107.0	18.0
15						
				fine sandy CLAY - orange-brown; moist; very stiff		
	6 - 3	◢	50/4"	silty fine SAND - orange & yellow-brown; dense	102.0	20.0
20						
25						
30						
35						

No groundwater encountered.
Bottom of boring at 19.5 feet
Drilled on 04/05/01
Logged by dd
Minute Man portable drilling rig
Modified California sampler
70# hammer

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Figure A6 - Log of Boring GF-6

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				6" CONCRETE SLAB SAND - buff; loose; very moist/wet (excavated prior to drilling)		
5	7 - 1	◢	10	sandy CLAY with few gravels - dark grey-brown; saturated; medium stiff		
10						
	7 - 2	◢	70	CLAY - mottled orange & grey-brown; slightly moist; hard		
15						
	SPT 7 - 3	◢	43	as above; very stiff		
20						
25						
30						
35						

Groundwater encountered at 5 feet.
Bottom of boring at 19.5 feet
Drilled on 05/09/01
Logged by BA
Mobile B-24 drilling rig
Modified California & Split Spoon samplers
140# hammer

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Figure A - Log of Boring GF-7

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				6" CONCRETE SLAB		
				SAND		
				CONCRETE		
5						
	8 - 1	◢	8	CLAY - dark grey & black; moist; medium stiff		
10						
	8 - 2	◢	15	sandy CLAY - grey; moist; stiff		
15						
	no recovery	◺	31	sandy CLAY - very stiff		
20						
25						
30						
35						

Groundwater encountered at 7 feet
Bottom of boring at 19.5 feet
Drilled on 05/09/01
Logged by ba
Mobile B-24 drilling rig
Modified California sampler
140# hammer

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Figure A - Log of Boring GF-8

LOG OF BORING

DEPTH (ft)	SAMPLE NO.	SAMPLE LOC.	BLOW COUNTS (12 inches)	DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)
				6" CONCRETE		
				CONCRETE SLAB		
5 10				Materials removed by Levine-Fricke prior to GeoForensics drilling.		
	9 - 1	◢	46	CLAY with sand - mottled orange & grey-brown; slightly moist; very stiff		
15 20 25 30 35				Groundwater reported at 5 feet. Bottom of boring at 14.5 feet Drilled on 05/09/01 Logged by ba Mobile B-24 drilling rig Modified California sampler 140# hammer		

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Figure A - Log of Boring GF-9

COOPER TESTING LABS

MOISTURE DENSITY - POROSITY DATA SHEET

Job #	060-1192A
Client	Geoforensics
Project/Location	Avalon
Date	4/25/01

Boring #	2-3	2-3	4-1A	4-1A	
Depth (ft)	20.5	20.5	4	4	
Soil Type	light brown sandy CLAY (initial)	light brown sandy CLAY (initial)	brown sandy CLAY (initial)	light brown sandy CLAY (initial)	
Specific Gravity	2.75 ASSUMED	2.75 ASSUMED	2.75 ASSUMED	2.75 ASSUMED	
Volume Total cc	75.063	68.382	75.063	66.356	
Volume of Solids	44.454	44.451	40.945	40.980	
Volume of Voids	30.609	23.931	34.118	25.376	
Void Ratio	0.689	0.538	0.833	0.619	
Porosity %	40.8%	35.0%	45.5%	38.2%	
Saturation %	97.9%	100.1%	83.8%	98.1%	
Moisture %	24.5%	19.6%	25.4%	22.1%	
Dry Density (pcf)	101.7	111.6	93.6	106.0	

Remarks

CRS test moisture & densities before and after test.

COOPER TESTING LABS

MOISTURE DENSITY - POROSITY DATA SHEET

Job #	060-1192B
Client	Geoforensics
Project/Location	Avalon
Date	4/25/01

Boring #	4-5	4-5	4-3	4-3	
Depth (ft)	24	24	14.5	14.5	
Soil Type	olivegray clayey SAND w/ gravel (initial)	olivegray clayey SAND w/ gravel (initial)	olivegray clayey SAND w/ gravel (initial)	olivegray clayey SAND w/ gravel (initial)	
Specific Gravity	2.70 ASSUMED	2.70 ASSUMED	2.75 ASSUMED	2.75 ASSUMED	
Volume Total cc	75.063	67.557	75.063	71.835	
Volume of Solids	48.588	47.881	51.876	51.788	
Volume of Voids	26.475	19.676	23.187	20.047	
Void Ratio	0.545	0.411	0.447	0.387	
Porosity %	35.3%	29.1%	30.9%	27.9%	
Saturation %	89.2%	99.2%	97.2%	95.2%	
Moisture %	18.0%	15.1%	15.8%	13.4%	
Dry Density (pcf)	109.1	119.5	118.6	123.8	

Remarks

CRS test moisture & densities before & after test.

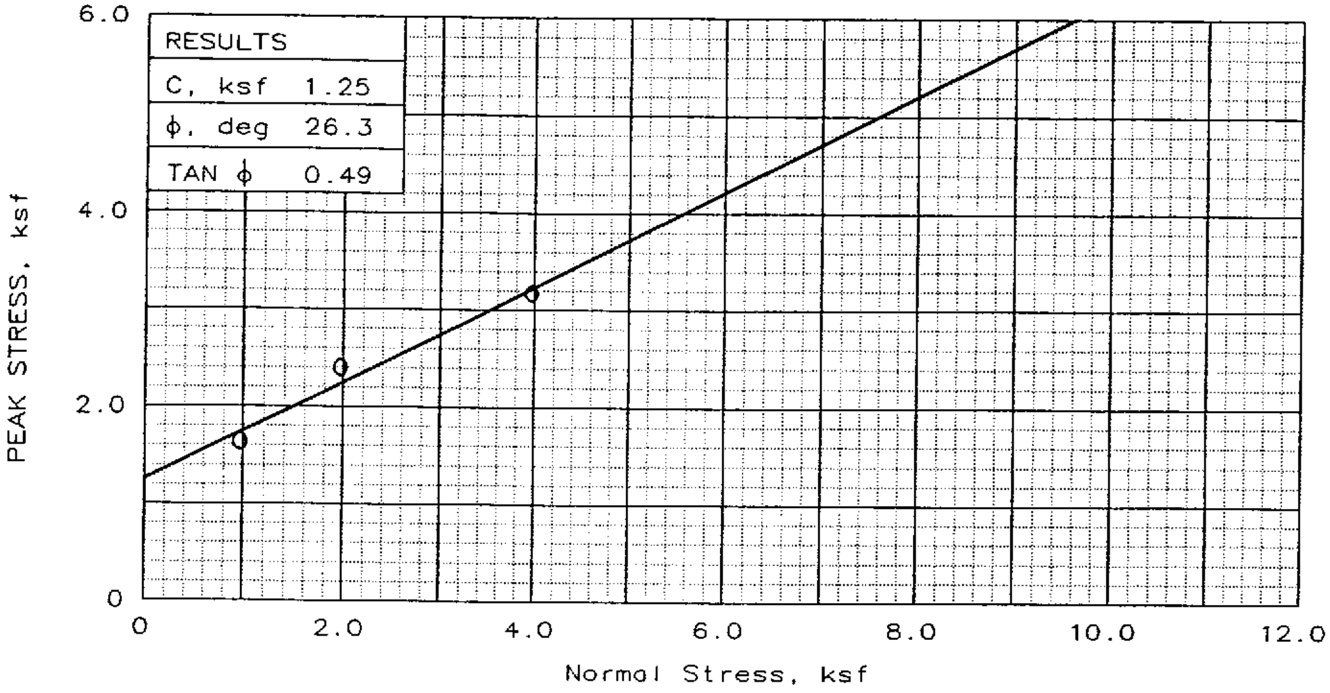
COOPER TESTING LABS

MOISTURE DENSITY - POROSITY DATA SHEET

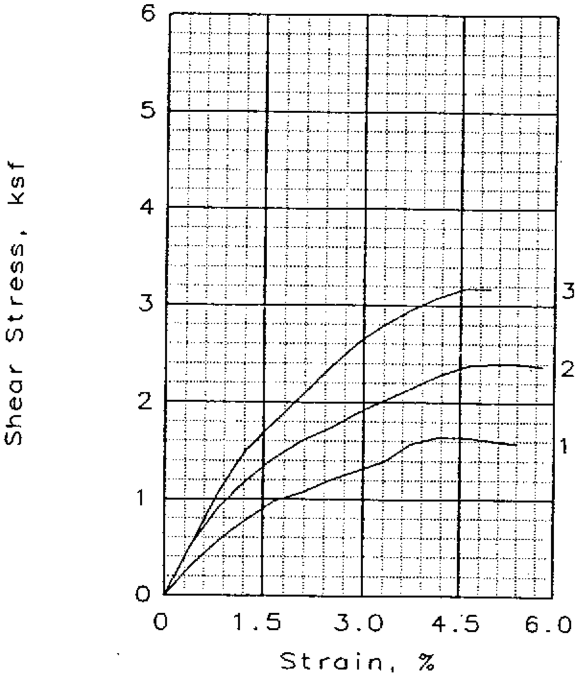
Job # Client Project/Location Date	060-1192C GeoForensics 201071 / Avalon 04/26/01

Boring #	6-1				
Depth (ft)	6.5				
Soil Type	brown clayey SAND with gravel				
Specific Gravity	2.70 ASSUMED				
Volume Total cc	406.774				
Volume of Solids	229.138				
Volume of Voids	177.636				
Void Ratio	0.775				
Porosity %	43.7%				
Saturation %	50.8%				
Moisture %	14.6%				
Dry Density (pcf)	94.9				

Remarks



RESULTS	
C, ksf	1.25
ϕ, deg	26.3
TAN ϕ	0.49



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	25.7	26.7	25.1
	DRY DENSITY, pcf	98.2	97.6	99.6
	SATURATION, %	94.4	94.5	93.2
	VOID RATIO	0.748	0.790	0.756
	DIAMETER, in	2.41	2.41	2.42
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	26.1	26.9	24.8
	DRY DENSITY, pcf	99.3	99.3	103.1
	SATURATION, %	98.5	98.8	99.8
	VOID RATIO	0.729	0.761	0.696
	DIAMETER, in	2.41	2.41	2.42
	HEIGHT, in	0.99	0.98	0.97
NORMAL STRESS, ksf		1.00	2.00	4.00
PEAK STRESS, ksf		1.64	2.40	3.18
STRAIN, %		4.1	5.0	4.6
Staged, ksf				
STRAIN, %				
Strain rate, %/min		1.00	1.00	1.00

SAMPLE TYPE: Undisturbed
DESCRIPTION: olive brn., orange
CLAY with fine sand

ASSUMED SPECIFIC GRAVITY= 2.75
REMARKS: **DS-CU**
An undrained condition cannot
be completely accomplished.

Fig. No.: ______

CLIENT: GeoForensics

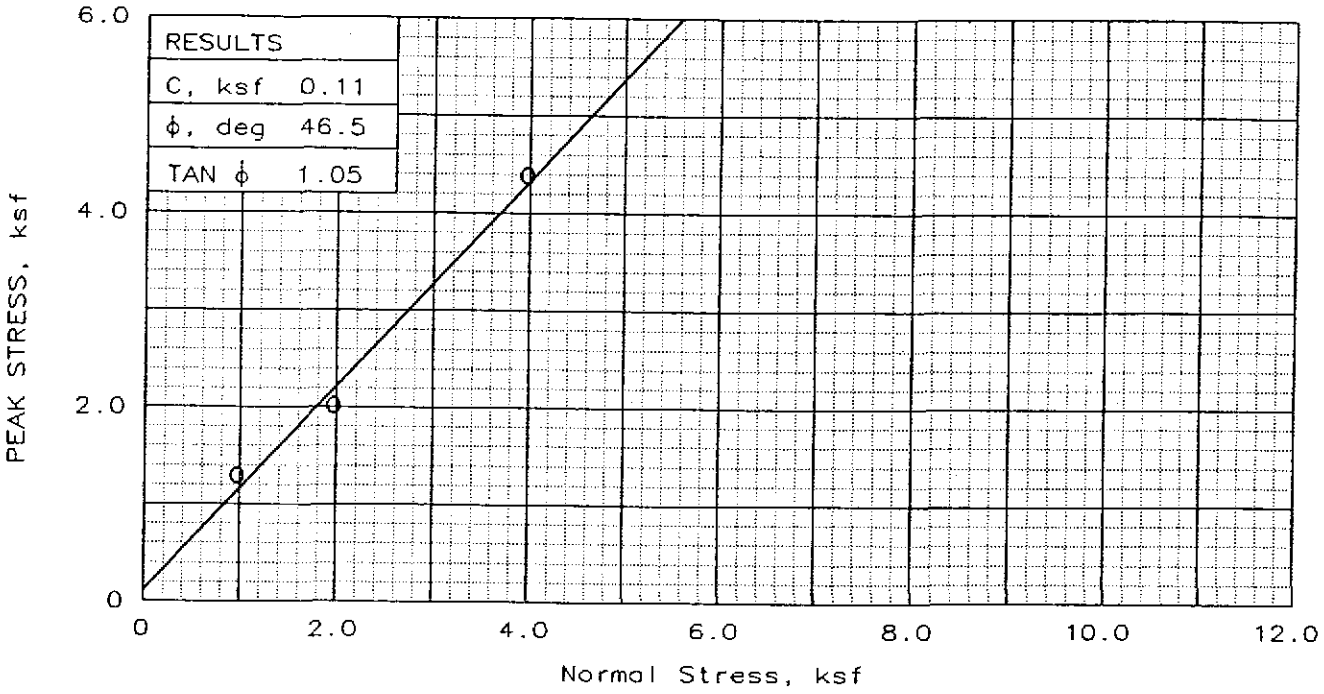
PROJECT: 201071 / Avalon

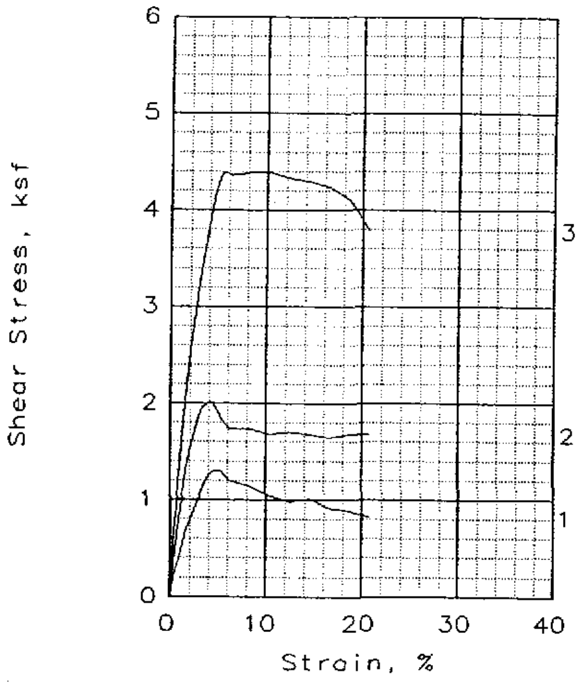
SAMPLE LOCATION: 1-1 @ 4'

PROJ. NO.: 060-1192 DATE: 4/17/01

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	23.1	22.4	19.8
	DRY DENSITY, pcf	103.1	104.0	108.3
	SATURATION, %	98.2	97.4	93.2
	VOID RATIO	0.634	0.621	0.585
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	22.3	21.3	19.0
	DRY DENSITY, pcf	105.0	106.7	112.1
	SATURATION, %	99.7	99.1	98.2
	VOID RATIO	0.605	0.580	0.531
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	0.98	0.97	0.97
NORMAL STRESS, ksf		1.00	2.00	4.00
PEAK STRESS, ksf		1.29	2.02	4.39
STRAIN, %		4.6	4.1	5.4
Staged, ksf				
STRAIN, %				
Strain rate, %/min		1.00	1.00	1.00

SAMPLE TYPE: Undisturbed

DESCRIPTION: brown silty SAND with gravel

ASSUMED SPECIFIC GRAVITY= 2.7

REMARKS: **DS-CU**

An undrained condition cannot be completely accomplished.

Fig. No.: ______

CLIENT: GeoForeniscs

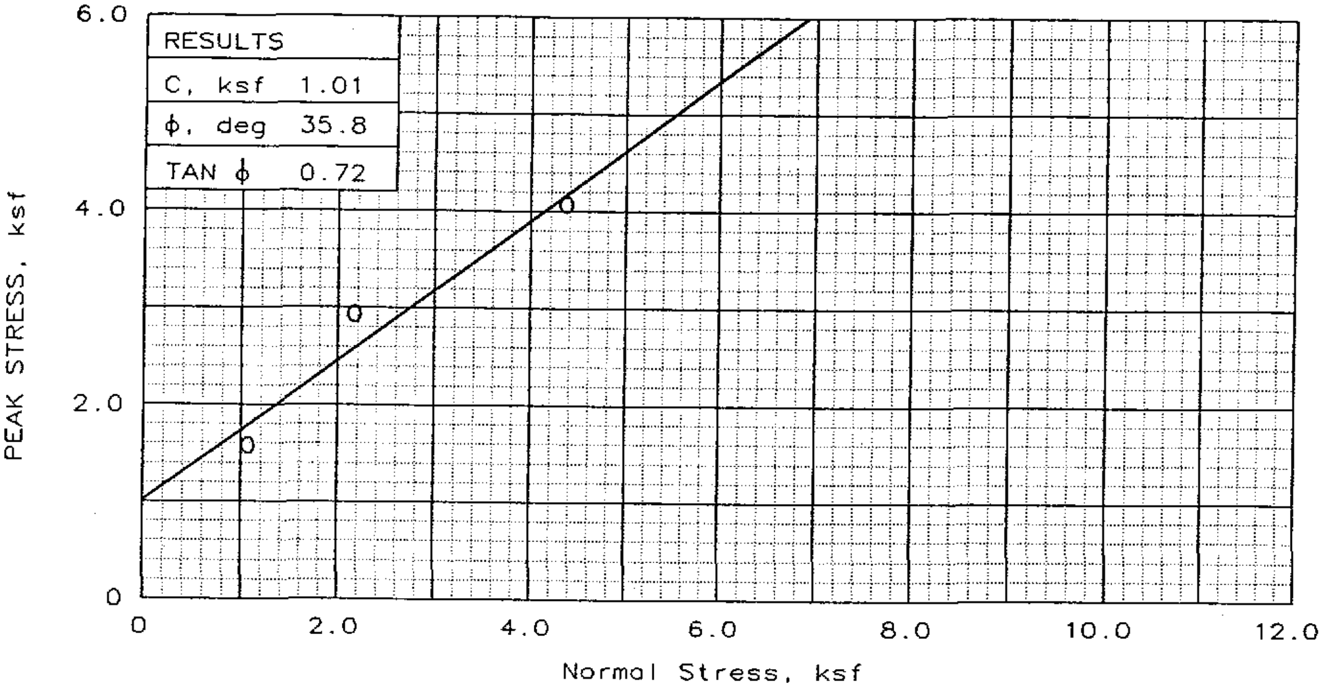
PROJECT: 201071 / Avalon

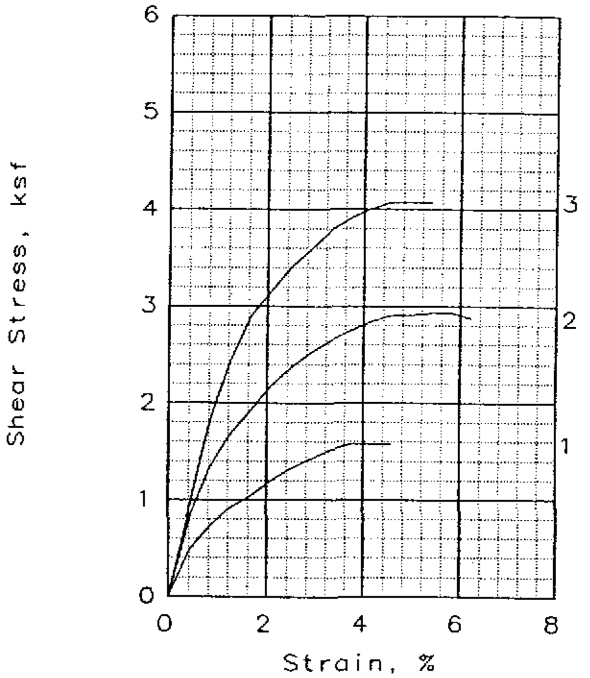
SAMPLE LOCATION: 1-2 @ 9.5'

PROJ. NO.: 60-1192A DATE: 4/17/01

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	18.4	20.1	18.4
	DRY DENSITY, pcf	110.1	108.7	111.0
	SATURATION, %	90.6	92.7	89.4
	VOID RATIO	0.560	0.608	0.575
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	19.0	20.7	18.4
	DRY DENSITY, pcf	112.1	110.5	114.5
	SATURATION, %	98.5	99.3	97.7
	VOID RATIO	0.532	0.583	0.526
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	0.98	0.98	0.97
NORMAL STRESS, ksf		1.10	2.20	4.40
PEAK STRESS, ksf		1.58	2.94	4.07
STRAIN, %		3.7	5.4	4.6
Staged, ksf				
STRAIN, %				
Strain rate, %/min		1.00	1.00	1.00

SAMPLE TYPE: Undisturbed
DESCRIPTION: gray clayey SAND

ASSUMED SPECIFIC GRAVITY= 2.75
REMARKS: **DS-CU**
An undrained condition cannot be completely accomplished.

Fig. No.: ______

CLIENT: GeoForensics

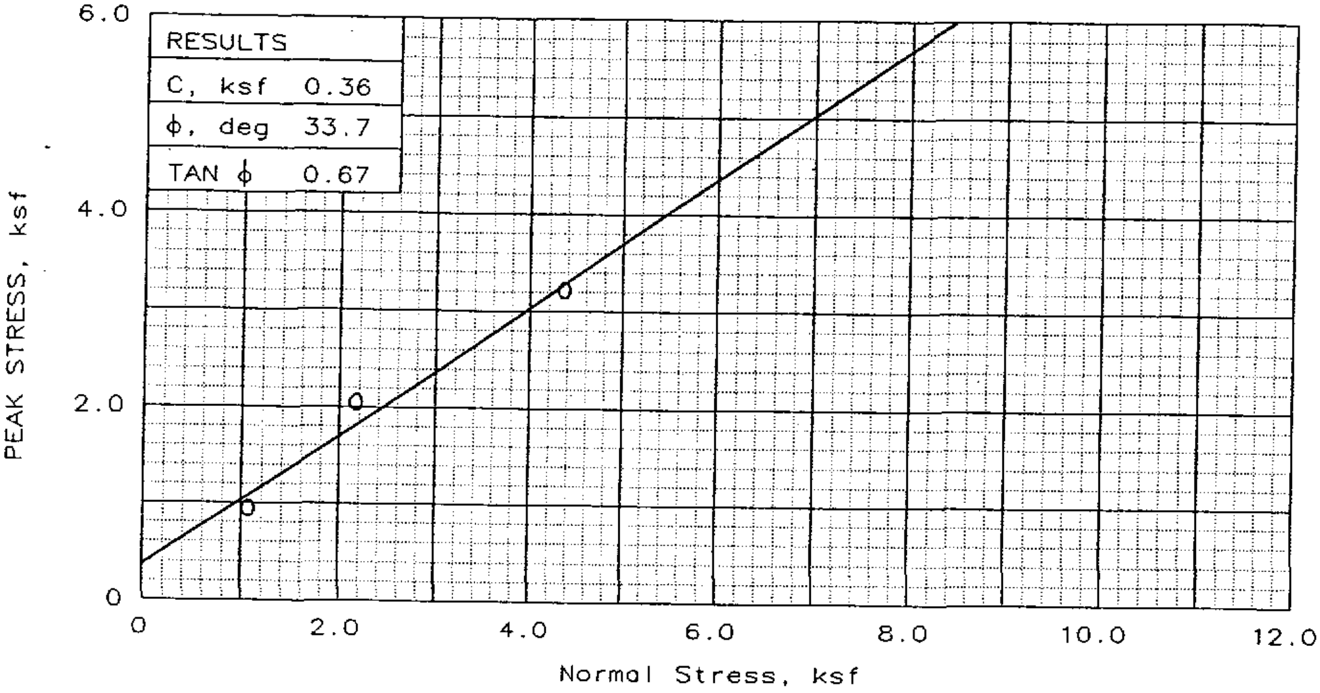
PROJECT: 201071 / Avalon

SAMPLE LOCATION: 1-4 @ 19.5'

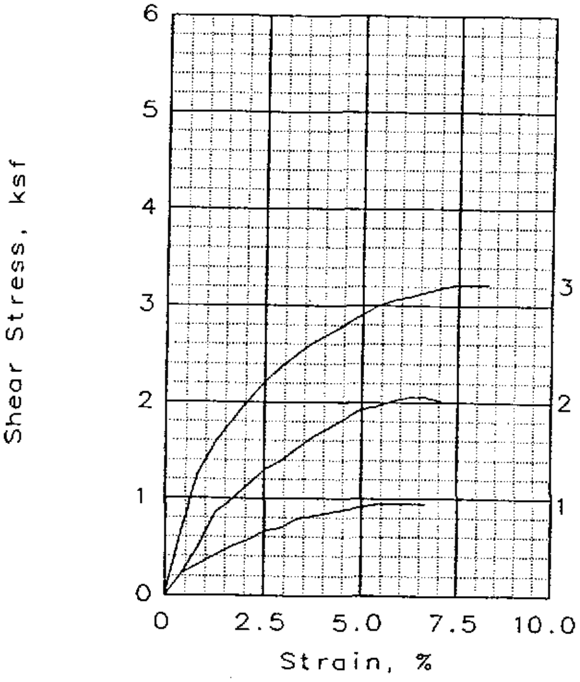
PROJ. NO.: 60-1192B DATE: 4/23/01

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY



RESULTS	
C, ksf	0.36
ϕ, deg	33.7
TAN ϕ	0.67



	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	28.6	24.5	23.9
	DRY DENSITY, pcf	90.8	97.7	99.1
	SATURATION, %	90.2	91.3	92.2
	VOID RATIO	0.857	0.725	0.700
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	28.0	23.2	21.9
	DRY DENSITY, pcf	94.1	103.0	105.8
	SATURATION, %	95.5	98.6	99.7
	VOID RATIO	0.790	0.637	0.593
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	0.96	0.95	0.94
NORMAL STRESS, ksf		1.10	2.20	4.40
PEAK STRESS, ksf		0.95	2.05	3.22
STRAIN, %		5.4	6.2	7.5
Staged, ksf				
STRAIN, %				
Strain rate, %/min		1.00	1.00	1.00

SAMPLE TYPE: Undisturbed
DESCRIPTION: drk. olive gray
clayey SAND

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: **DS-CU**
An undrained condition cannot
be completely accomplished.

Fig. No.: ______

CLIENT: GeoForensics

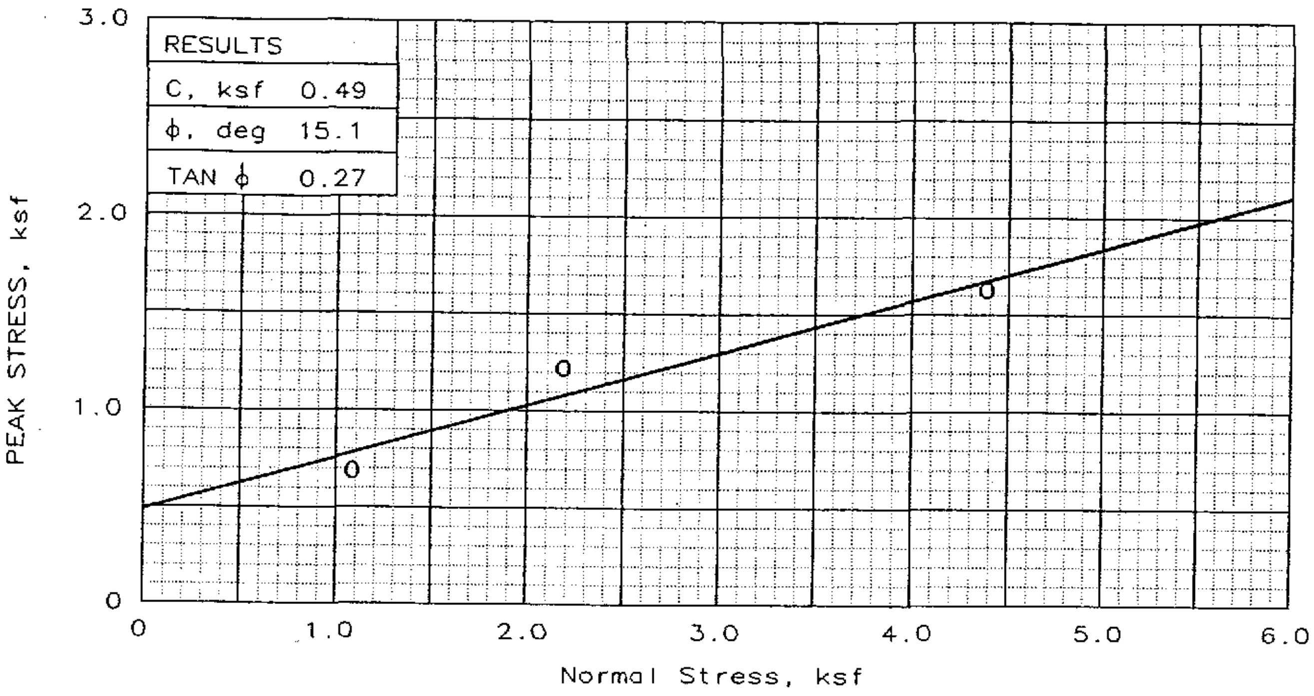
PROJECT: 201071 / Avalon

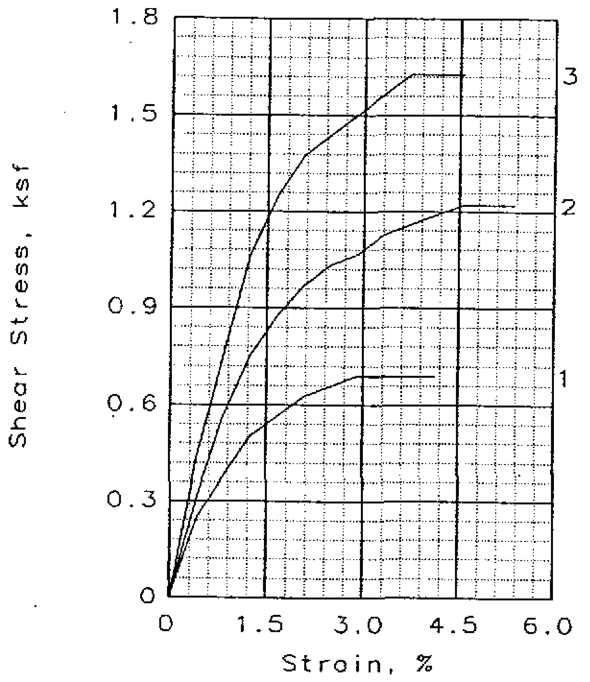
SAMPLE LOCATION: 2-18 @ 9.5'

PROJ. NO.: 60-1192C DATE: 4/23/01

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	22.3	22.4	24.3
	DRY DENSITY, pcf	98.1	98.8	98.1
	SATURATION, %	83.8	85.6	91.5
	VOID RATIO	0.719	0.706	0.717
	DIAMETER, in	2.42	2.42	2.42
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	23.0	22.4	21.3
	DRY DENSITY, pcf	100.6	102.6	105.1
	SATURATION, %	91.9	93.9	95.1
	VOID RATIO	0.675	0.643	0.604
	DIAMETER, in	2.42	2.42	2.42
	HEIGHT, in	0.97	0.96	0.93
NORMAL STRESS, ksf		1.10	2.20	4.40
PEAK STRESS, ksf		0.69	1.22	1.63
STRAIN, %		2.9	4.5	3.7
Ultimate Stress, ksf				
STRAIN, %				
Strain rate, %/min		1.00	1.00	1.00

SAMPLE TYPE: Undisturbed
DESCRIPTION: dark brown sandy CLAY

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: **DS-CU**
An undrained condition cannot be completely accomplished.

Fig. No.: ______

CLIENT: GeoForensics

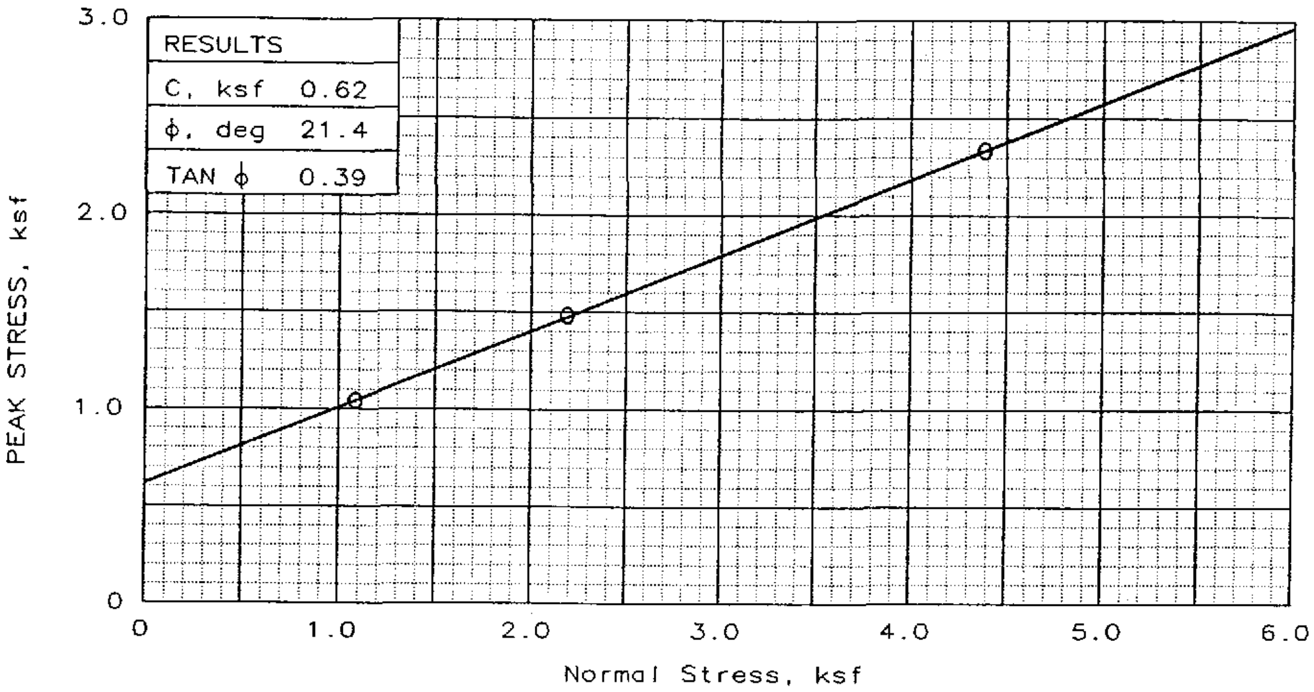
PROJECT: 201071 / Avalon

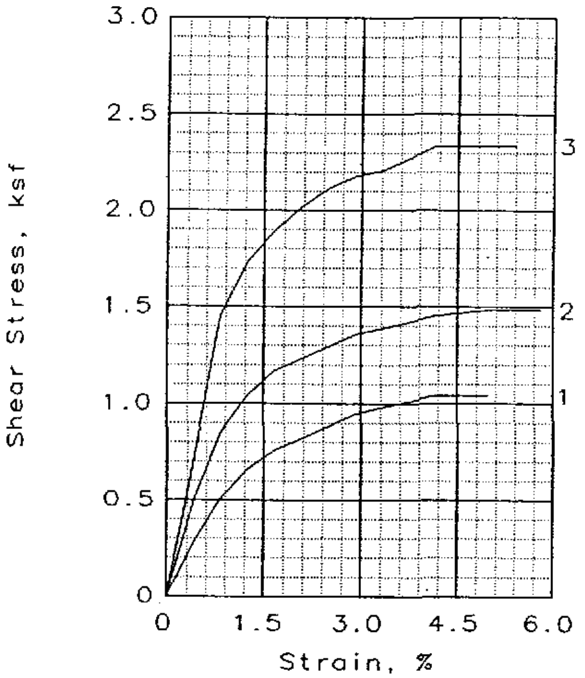
SAMPLE LOCATION: 4-1-B @ 4.5'

PROJ. NO.: 60-11920 DATE: 04/27/01

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	25.9	24.5	23.8
	DRY DENSITY, pcf	97.4	99.7	101.0
	SATURATION, %	95.7	93.2	91.2
	VOID RATIO	0.731	0.722	0.731
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	25.7	23.8	21.8
	DRY DENSITY, pcf	99.2	103.2	107.4
	SATURATION, %	99.4	98.5	97.3
	VOID RATIO	0.699	0.664	0.628
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	0.98	0.97	0.94
NORMAL STRESS, ksf		1.10	2.20	4.40
PEAK STRESS, ksf		1.04	1.48	2.34
STRAIN, %		4.1	5.0	4.1
Ultimate Stress, ksf				
STRAIN, %				
Strain rate, %/min		1.00	1.00	1.00

SAMPLE TYPE: Undisturbed
DESCRIPTION: black CLAY with
sand

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: **DS-CU**
An undrained condition cannot
be completely accomplished.

Fig. No.: ______

CLIENT: GeoForensics

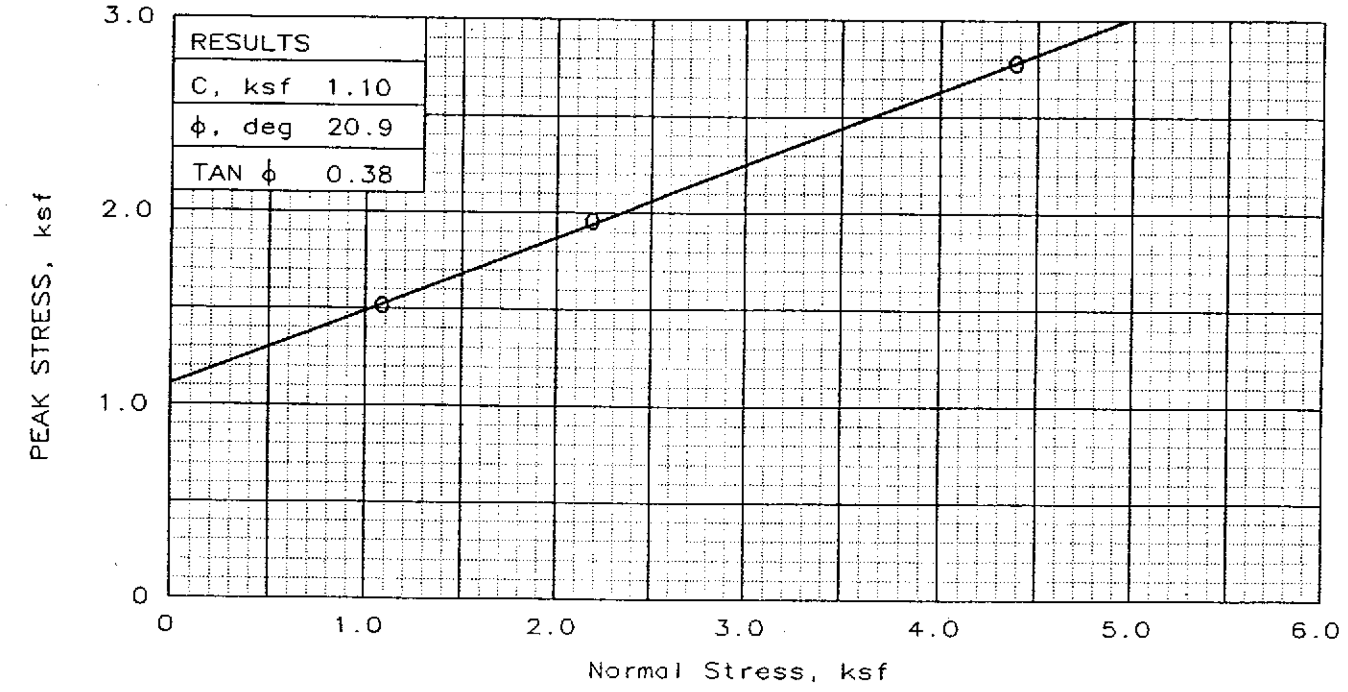
PROJECT: 201071 / Avalon

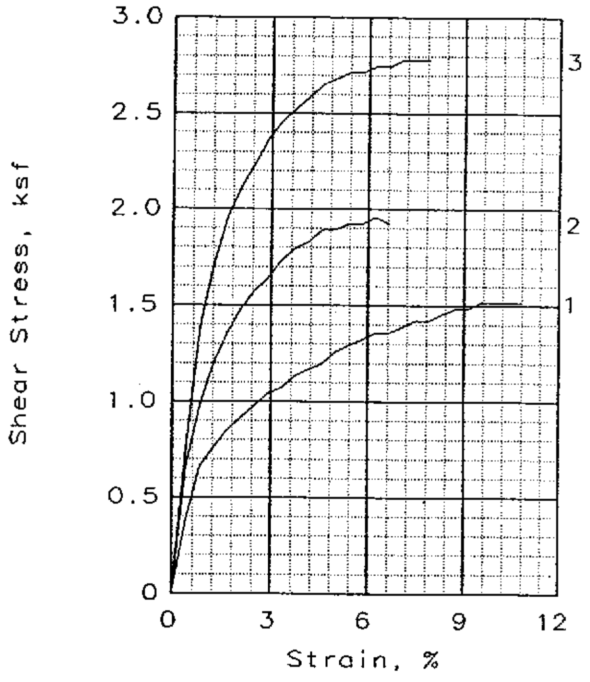
SAMPLE LOCATION: 4-2 @ 9.5'

PROJ. NO.: 60-1192E DATE: 4/27/01

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY





	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	19.3	19.5	18.9
	DRY DENSITY, pcf	107.8	108.4	108.9
	SATURATION, %	92.4	89.1	90.0
	VOID RATIO	0.563	0.613	0.576
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	19.4	19.0	17.7
	DRY DENSITY, pcf	109.7	112.9	115.2
	SATURATION, %	97.4	97.2	99.3
	VOID RATIO	0.537	0.548	0.490
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	0.98	0.96	0.95
NORMAL STRESS, ksf		1.10	2.20	4.40
PEAK STRESS, ksf		1.52	1.96	2.78
STRAIN, %		9.5	6.2	7.1
Ultimate Stress, ksf				
STRAIN, %				
Strain rate, %/min		1.00	1.00	1.00

SAMPLE TYPE: Undisturbed
DESCRIPTION: brown clayey SAND
with gravel

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: **DS-CU**
An undrained condition cannot
be completely accomplished.

Fig. No.: ______

CLIENT: GeoForensics

PROJECT: 60-1192H / Avalon

SAMPLE LOCATION: 4-4 @ 19.5'

PROJ. NO.: 60-1192H DATE: 04/30/01

DIRECT SHEAR TEST REPORT

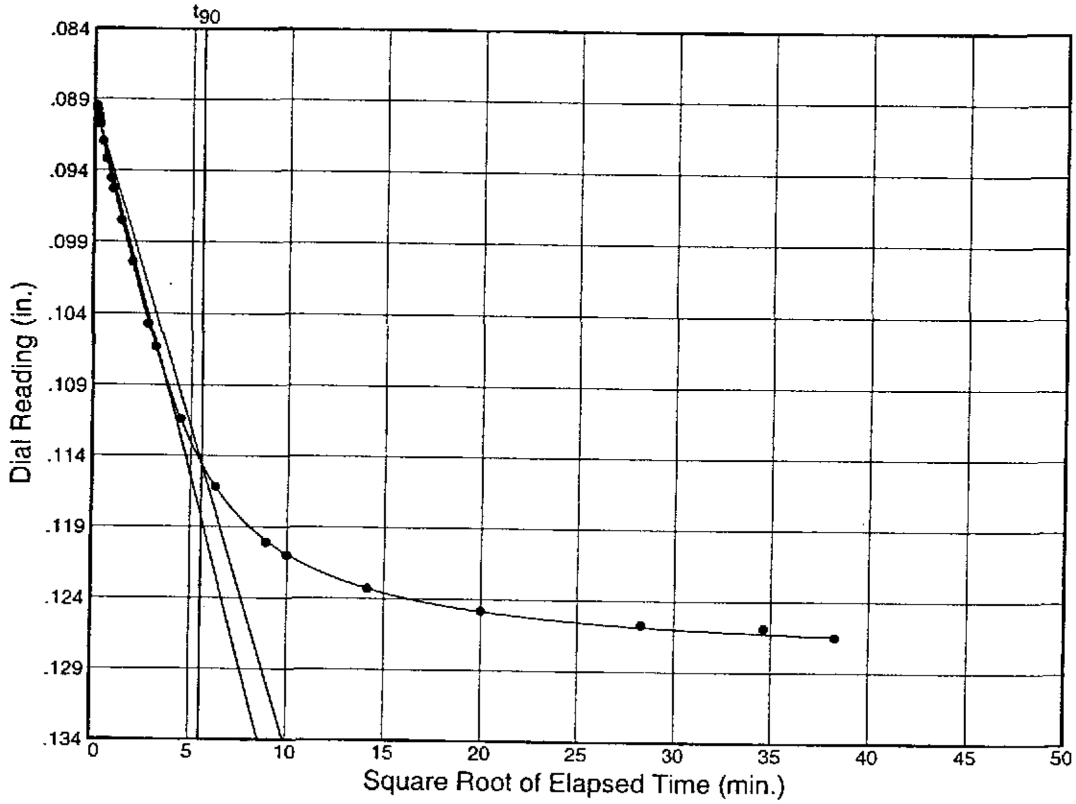
COOPER TESTING LABORATORY

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 2-1a

Elev./Depth: 9'



Load No.= 7

Load= 8.80 ksf

D_0 = 0.08958

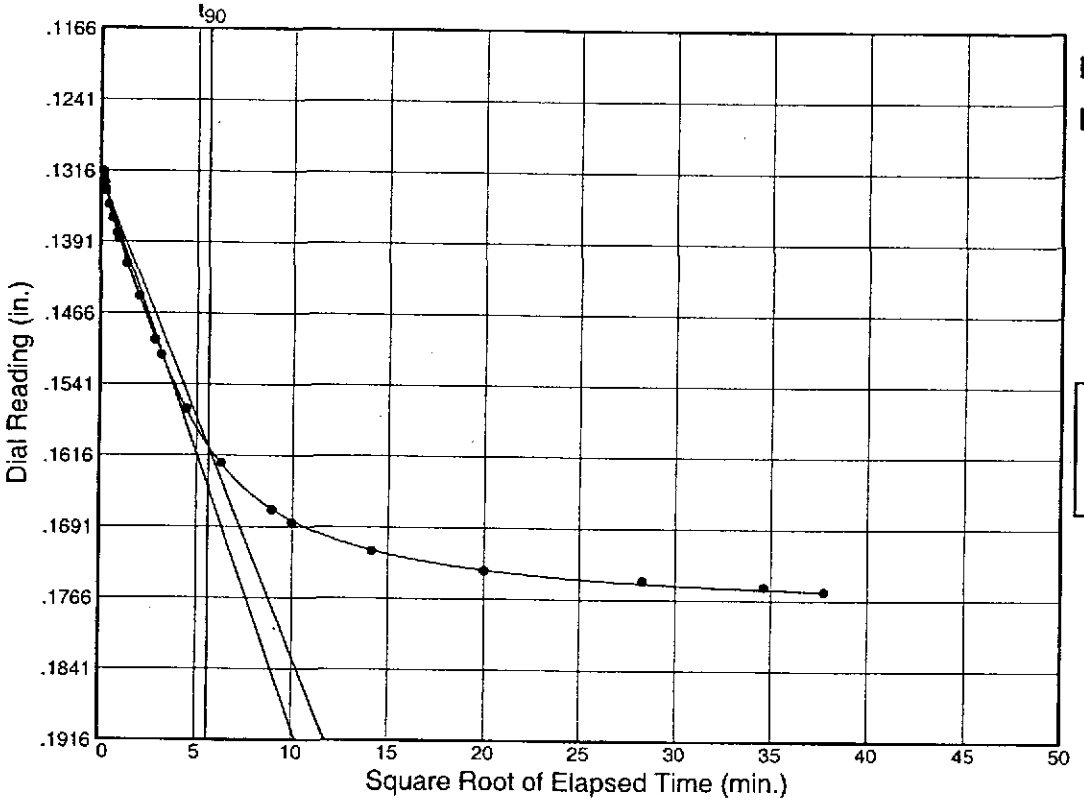
D_{90} = 0.11460

D_{100} = 0.11738

T_{90} = 31.05 min.

C_V @ T_{90}

0.05 ft.^{2}/day



Load No.= 8

Load= 17.60 ksf

D_0 = 0.13235

D_{90} = 0.16071

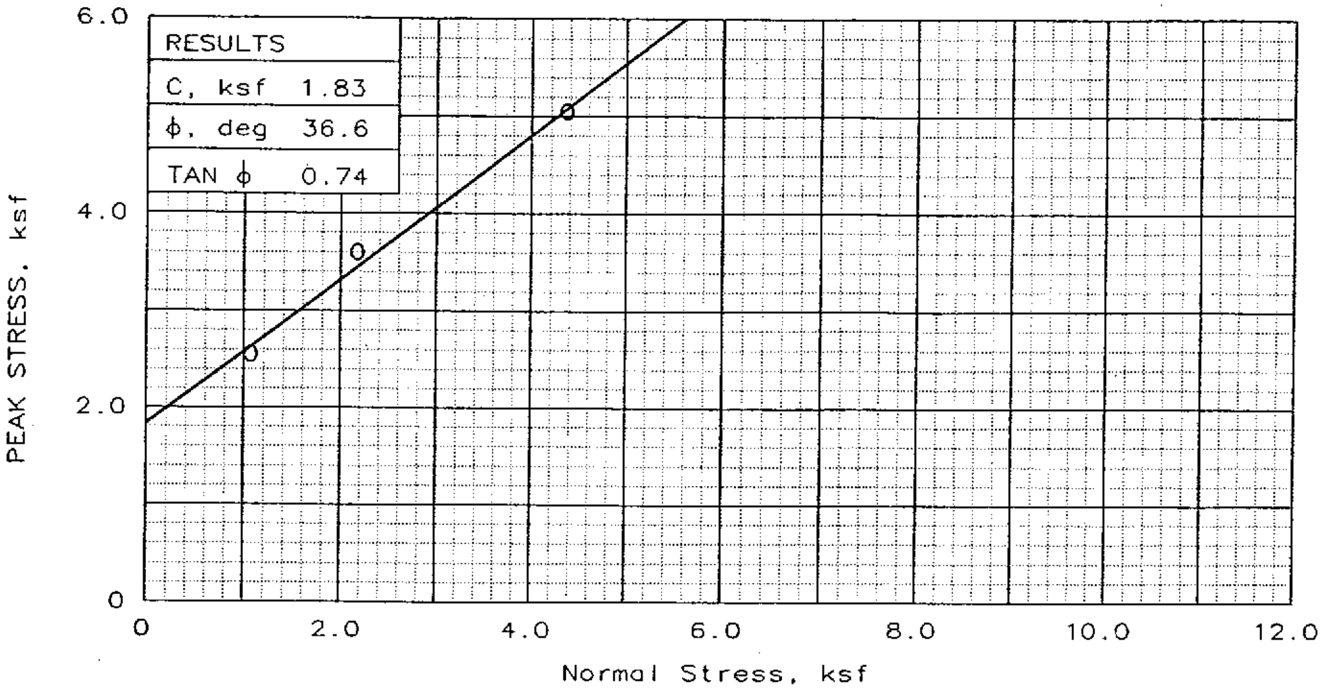
D_{100} = 0.16387

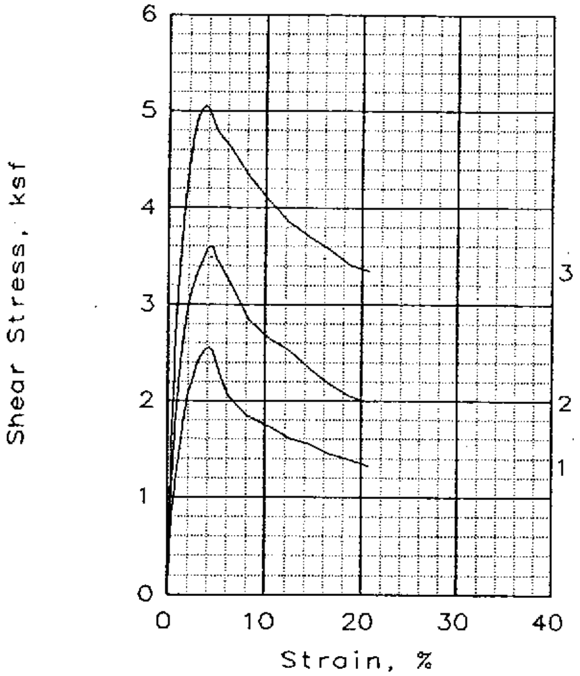
T_{90} = 31.59 min.

C_V @ T_{90}

0.05 ft.^{2}/day

Plate





	SAMPLE NO.:	1	2	3
INITIAL	WATER CONTENT, %	19.4	17.9	17.5
	DRY DENSITY, pcf	104.6	107.8	109.5
	SATURATION, %	80.9	80.6	82.0
	VOID RATIO	0.672	0.621	0.597
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	23.2	21.0	20.2
	DRY DENSITY, pcf	104.8	109.2	111.3
	SATURATION, %	97.4	97.9	99.1
	VOID RATIO	0.668	0.600	0.571
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	1.00	0.99	0.98
	NORMAL STRESS, ksf	1.10	2.20	4.40
	PEAK STRESS, ksf	2.56	3.60	5.05
	STRAIN, %	4.1	4.1	3.7
	Ultimate Stress, ksf			
	STRAIN, %			
	Strain rate, %/min	1.00	1.00	1.00

SAMPLE TYPE: Undisturbed
DESCRIPTION: brown sandy CLAY

ASSUMED SPECIFIC GRAVITY= 2.8
REMARKS: **DS-CU**
An undrained condition cannot be completely accomplished.

Fig. No.: ______

CLIENT: GeoForensics

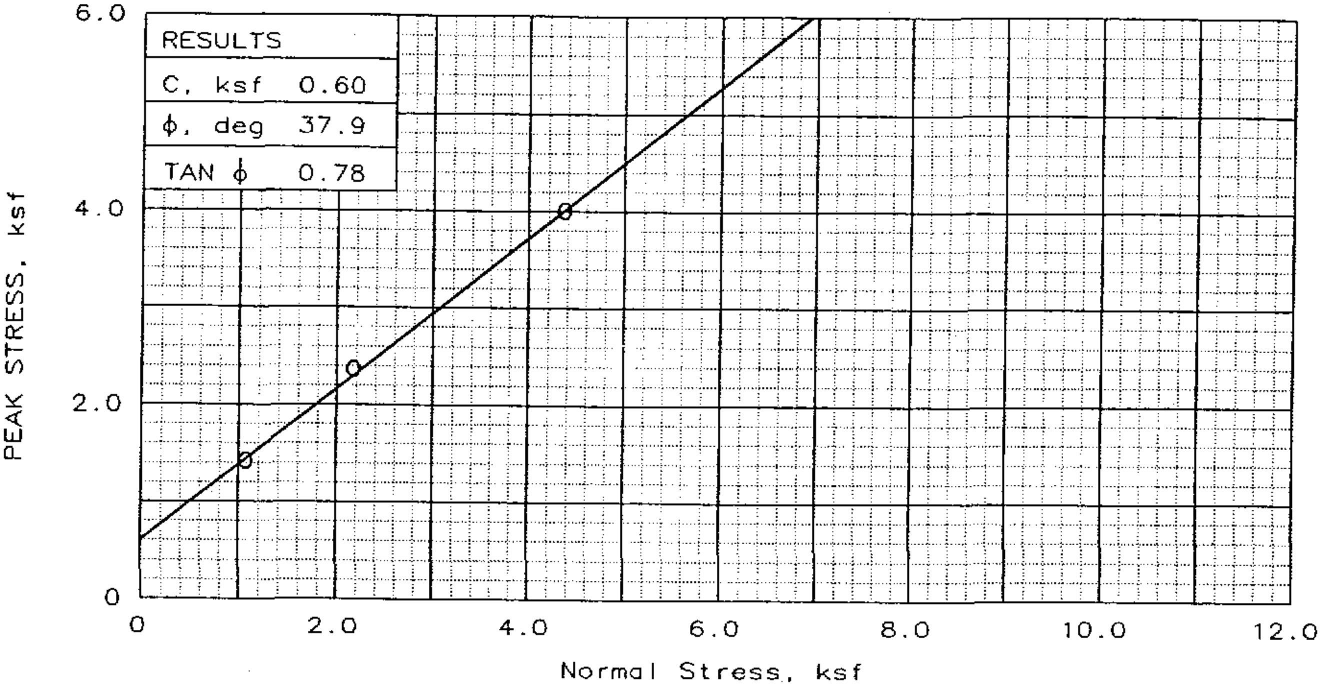
PROJECT: 201071 / Avalon

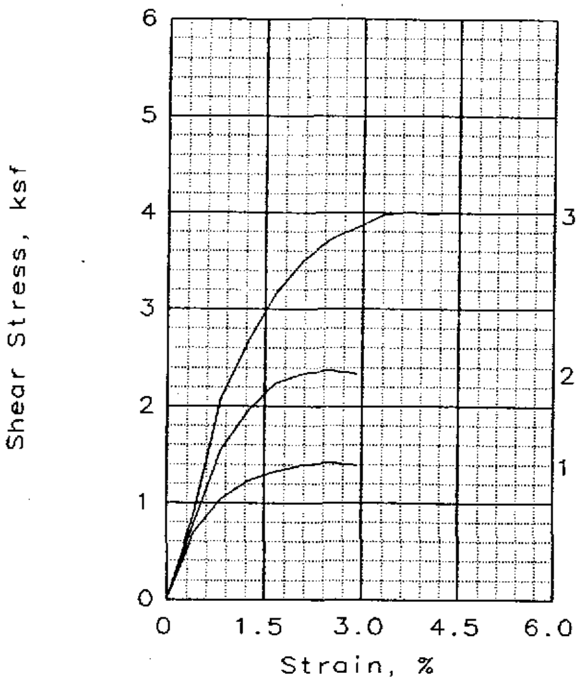
SAMPLE LOCATION: 6-2B @ 13'

PROJ. NO.: 60-1192F DATE: 04/27/01

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY





SAMPLE NO.:		1	2	3
INITIAL	WATER CONTENT, %	22.9	20.2	18.2
	DRY DENSITY, pcf	96.7	102.0	103.9
	SATURATION, %	83.2	83.6	76.6
	VOID RATIO	0.743	0.653	0.652
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	1.00	1.00	1.00
AT TEST	WATER CONTENT, %	26.7	23.2	21.3
	DRY DENSITY, pcf	97.7	103.0	107.3
	SATURATION, %	99.3	98.4	97.6
	VOID RATIO	0.726	0.636	0.600
	DIAMETER, in	2.41	2.41	2.41
	HEIGHT, in	0.99	0.99	0.97
NORMAL STRESS, ksf		1.10	2.20	4.40
PEAK STRESS, ksf		1.42	2.37	4.01
STRAIN, %		2.5	2.5	3.7
Ultimate Stress, ksf				
STRAIN, %				
Strain rate, %/min		1.00	1.00	1.00

SAMPLE TYPE: Undisturbed
DESCRIPTION: olive brn. mottled
orange sandy CLAY, tr. roots

ASSUMED SPECIFIC GRAVITY= 2.7
REMARKS: **DS-CU**
An undrained condition cannot
be completely accomplished.

Fig. No.: ______

CLIENT: GeoForensics

PROJECT: 201071 / Avalon

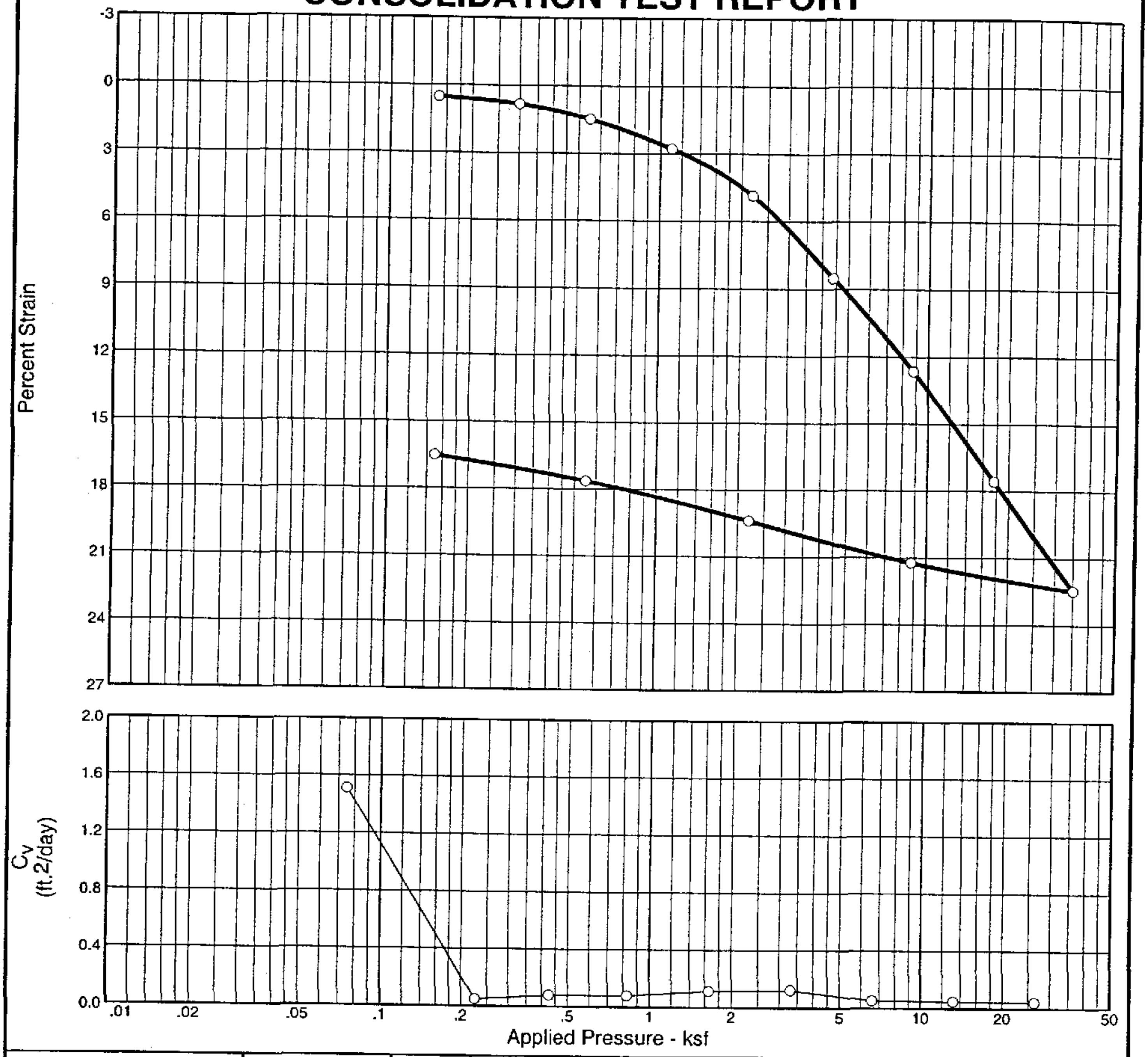
SAMPLE LOCATION: 6-3 @ 19'

PROJ. NO.: 60-1192G DATE: 04/30/01

DIRECT SHEAR TEST REPORT

COOPER TESTING LABORATORY

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
97.4 %	29.3 %	94.8			2.8			0.843

MATERIAL DESCRIPTION

olive brown CLAY w/sand

Project No. 060-1192 **Client:** Geoforensics

Project: Avalon

Source: 2-1a **Elev./Depth:** 9'

Remarks:

CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

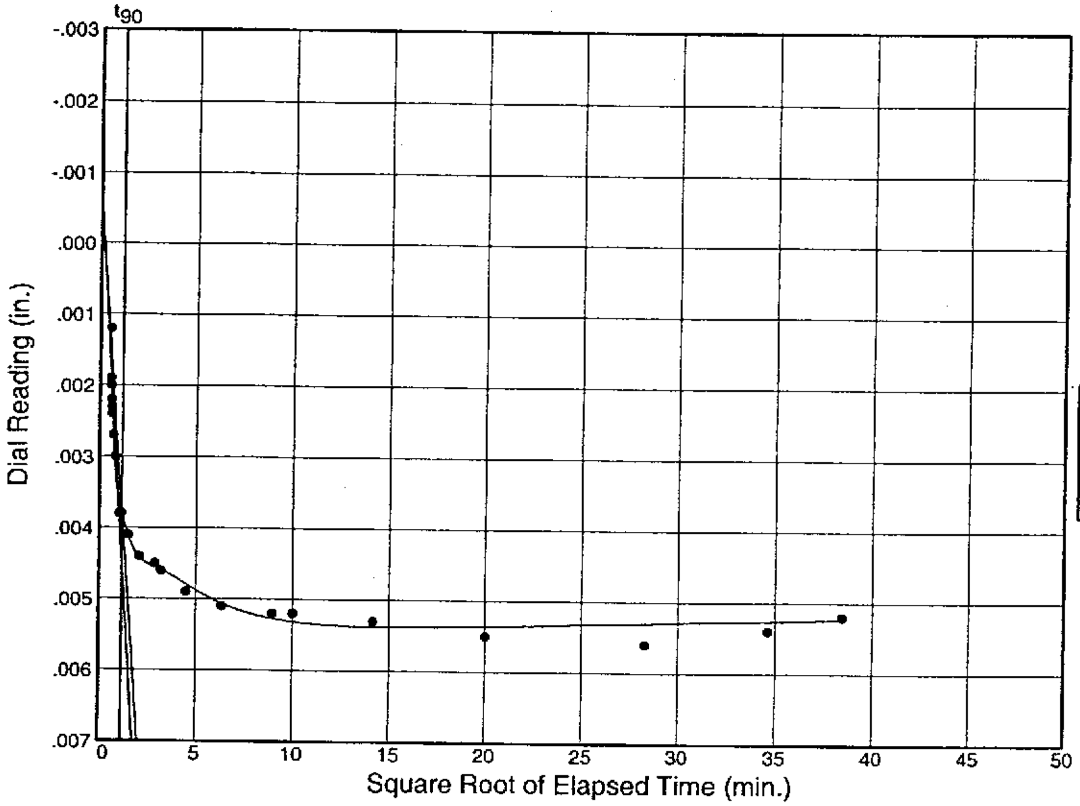
Plate

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 2-1a

Elev./Depth: 9'



Load No.= 1

Load= 0.15 ksf

D_0 = -0.00061

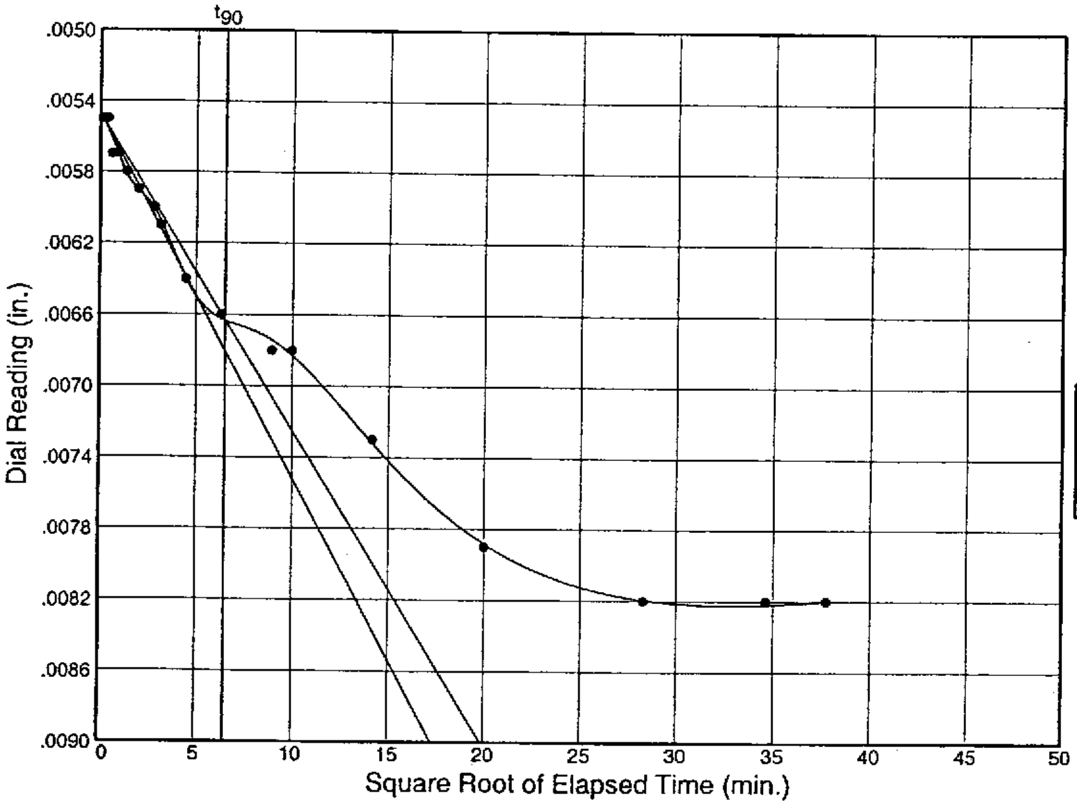
D_{90} = 0.00379

D_{100} = 0.00428

T_{90} = 1.40 min.

C_V @ T_{90}

1.51 ft.^{2}/day



Load No.= 2

Load= 0.30 ksf

D_0 = 0.00548

D_{90} = 0.00663

D_{100} = 0.00676

T_{90} = 42.59 min.

C_V @ T_{90}

0.05 ft.^{2}/day

COOPER TESTING LABORATORY

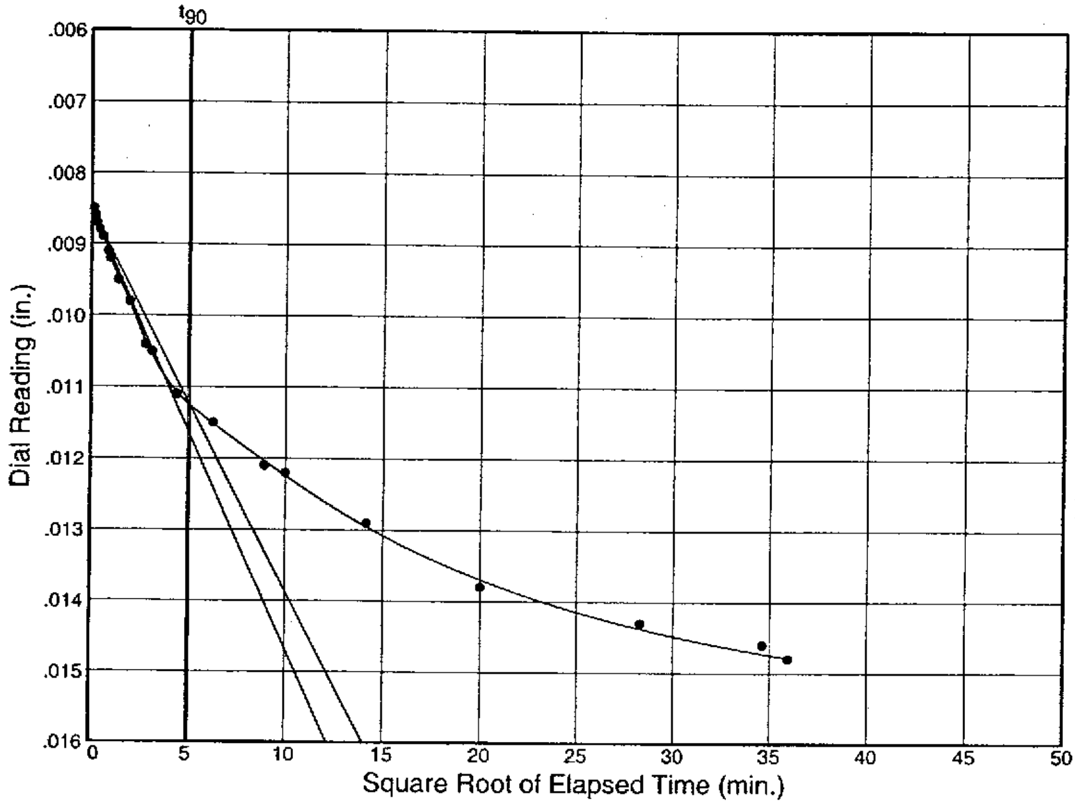
Plate

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 2-1a

Elev./Depth: 9'



Load No.= 3

Load= 0.55 ksf

D_0 = 0.00853

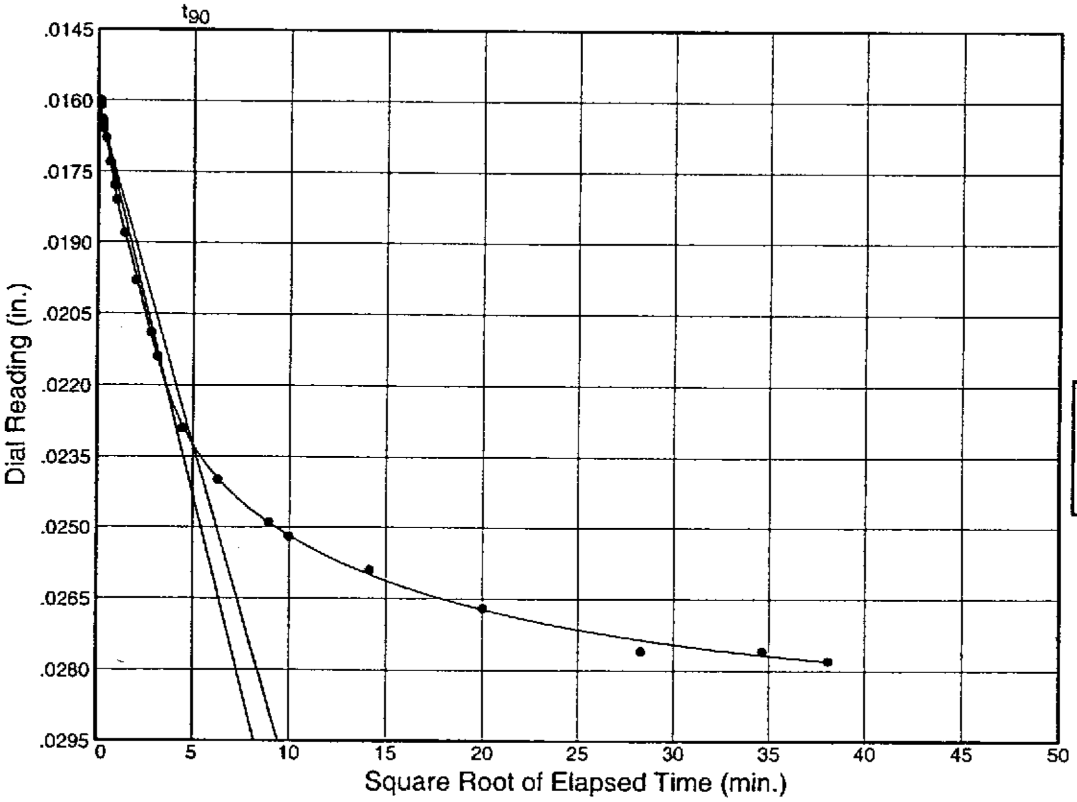
D_{90} = 0.01125

D_{100} = 0.01156

T_{90} = 26.08 min.

C_v @ T_{90}

0.08 ft.^{2}/day



Load No.= 4

Load= 1.10 ksf

D_0 = 0.01617

D_{90} = 0.02325

D_{100} = 0.02404

T_{90} = 25.17 min.

C_v @ T_{90}

0.08 ft.^{2}/day

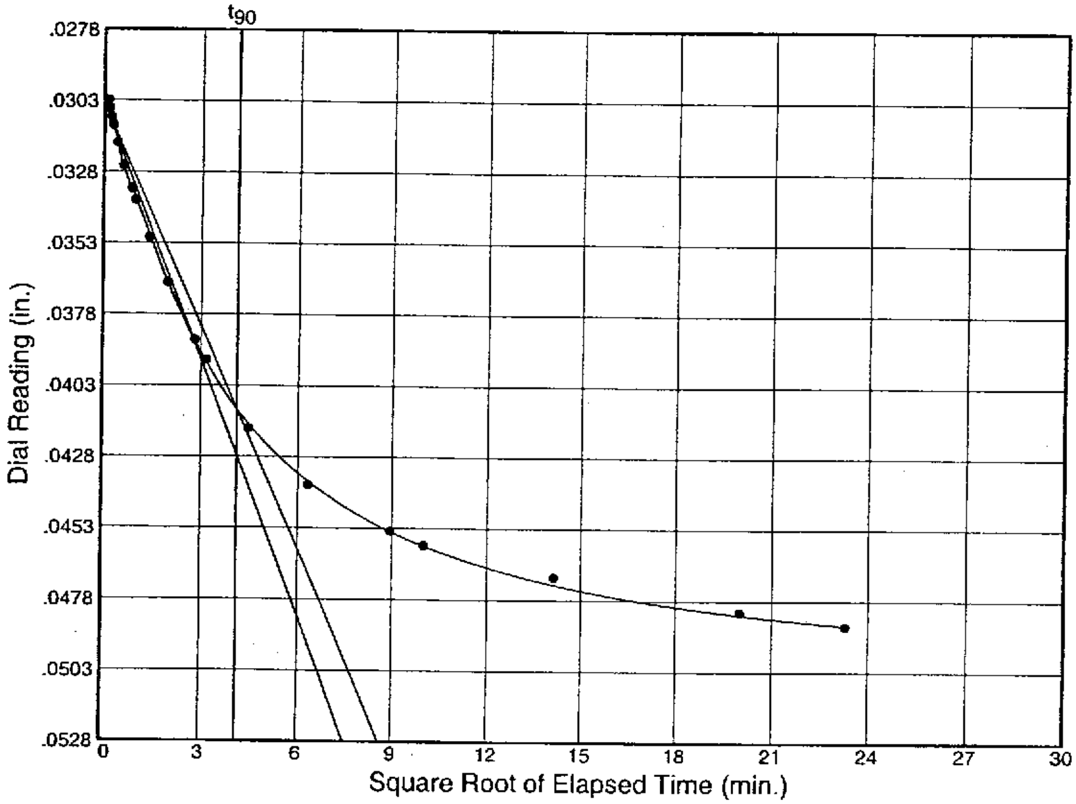
Plate

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 2-1a

Elev./Depth: 9'



Load No.= 5

Load= 2.20 ksf

D_0 = 0.03043

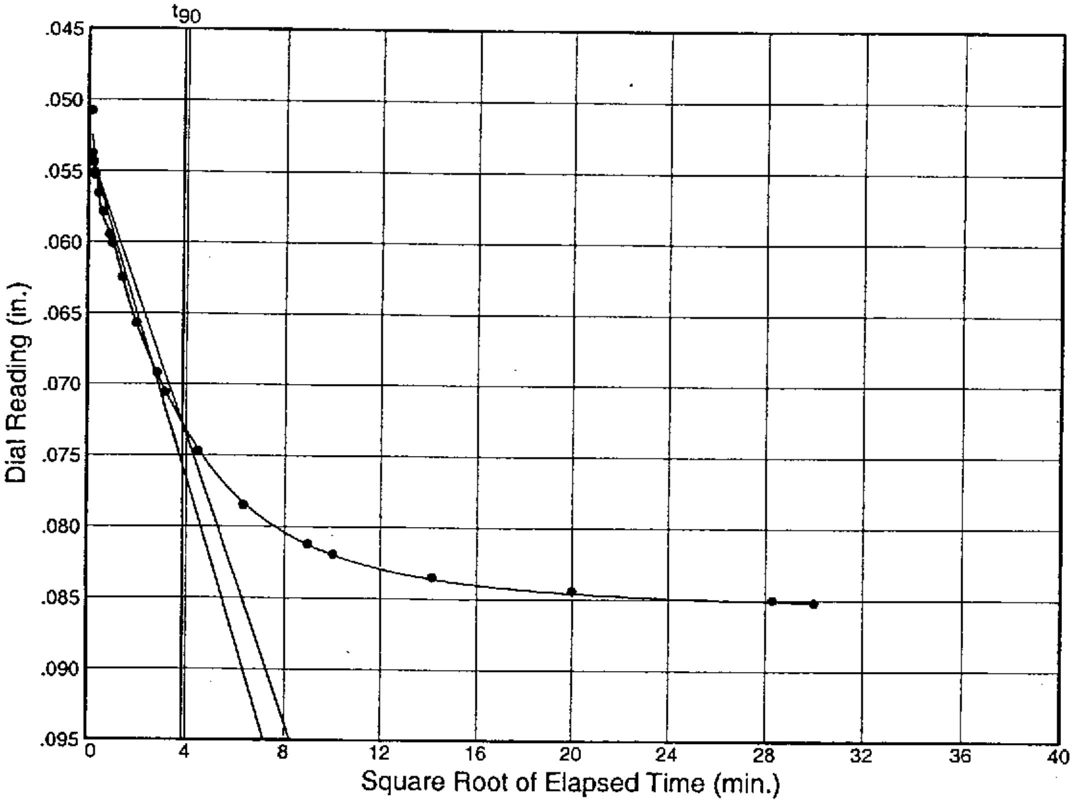
D_{90} = 0.04115

D_{100} = 0.04234

T_{90} = 17.01 min.

C_V @ T_{90}

0.12 ft.^{2}/day



Load No.= 6

Load= 4.40 ksf

D_0 = 0.05337

D_{90} = 0.07283

D_{100} = 0.07499

T_{90} = 14.85 min.

C_V @ T_{90}

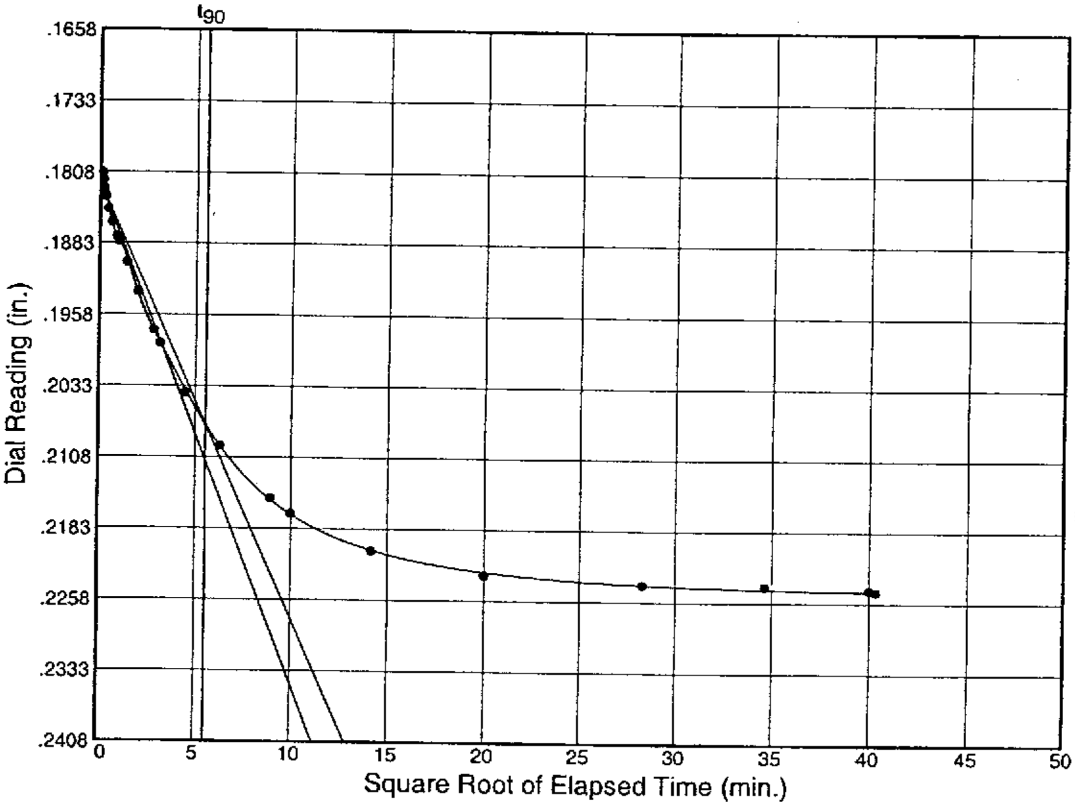
0.12 ft.^{2}/day

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 2-1a

Elev./Depth: 9'



Load No.= 9

Load= 35.20 ksf

D_0 = 0.18203

D_{90} = 0.20741

D_{100} = 0.21024

T_{90} = 30.88 min.

C_v @ T_{90}

0.04 ft.^{2}/day

COOPER TESTING LABORATORY

Plate

CONSOLIDATION TEST DATA

Client: Geoforensics
Project: Avalon
Project Number: 060-1192

Sample Data

Source: 2-1a
Sample No.:
Elev. or Depth: 9′
Sample Length (in./cm.):
Location:
Description: olive brown CLAY w/sand
Liquid Limit:
Plasticity Index:
USCS:
AASHTO:
Figure No.:
Testing Remarks:

Test Specimen Data

TOTAL SAMPLE
Wet w+t = 148.10 g.
Dry w+t = 114.50 g.
Tare Wt. = .00 g.
Height = 1.00 in.
Diameter = 2.42 in.
Weight = 148.10 g.

Moisture = 29.3 %
Wet Den. = 122.7 pcf
Dry Den. = 94.8 pcf

BEFORE TEST
Consolidometer # = 1

Spec. Gravity = 2.8
Height = 1.00 in.
Diameter = 2.42 in.
Defl. Table = n/a

Ht. Solids = 0.5425 in.
Dry Wt. = 114.50 g.*
Void Ratio = 0.843
Saturation = 97.4 %

AFTER TEST
Wet w+t = 141.30 g.
Dry w+t = 114.50 g.
Tare Wt. = .00 g.

Moisture = 23.4 %
Dry Wt. = 114.50 g.
Void Ratio = 0.539

*** Initial dry weight used in calculations**

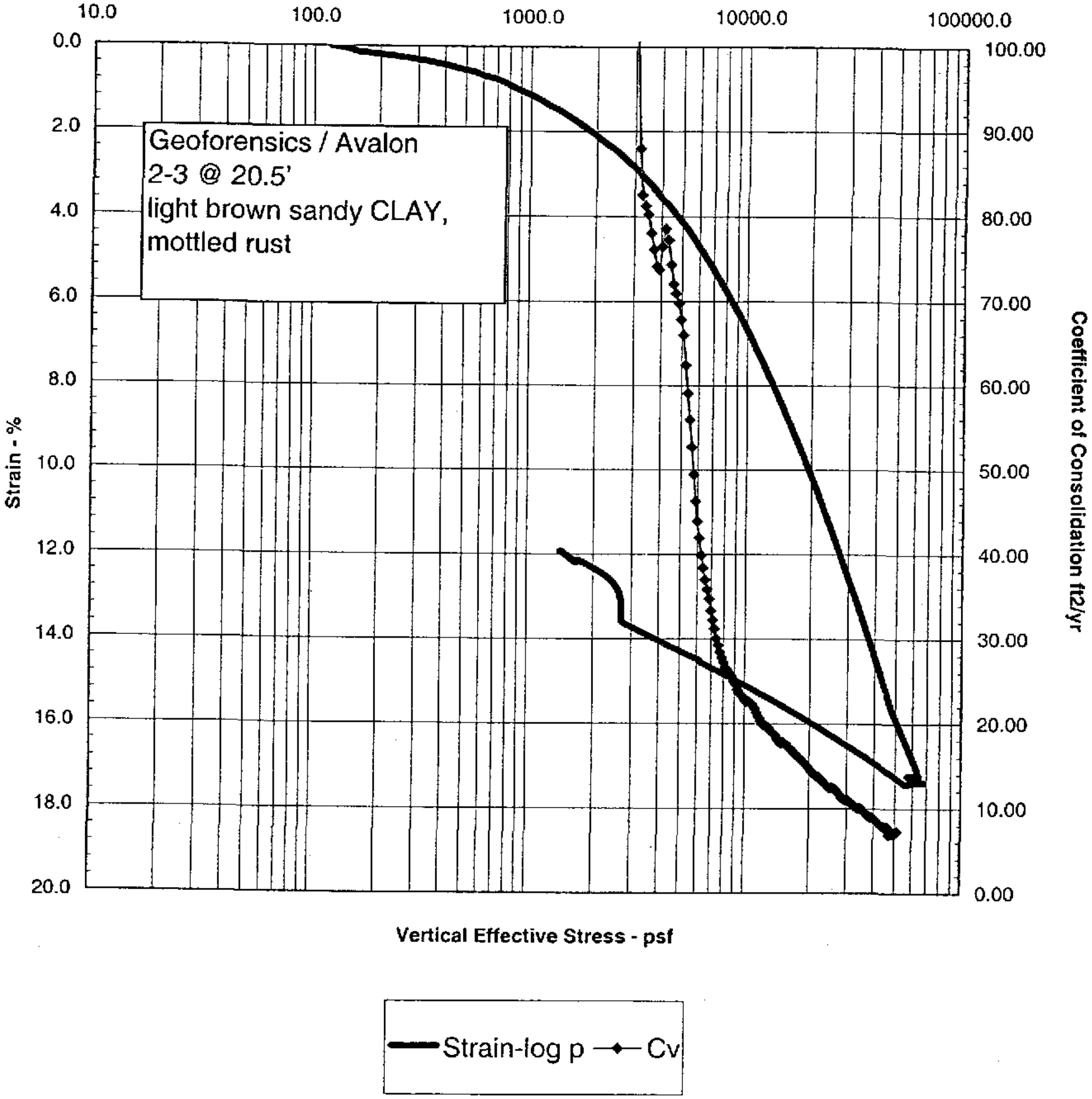
End-of-Load Summary

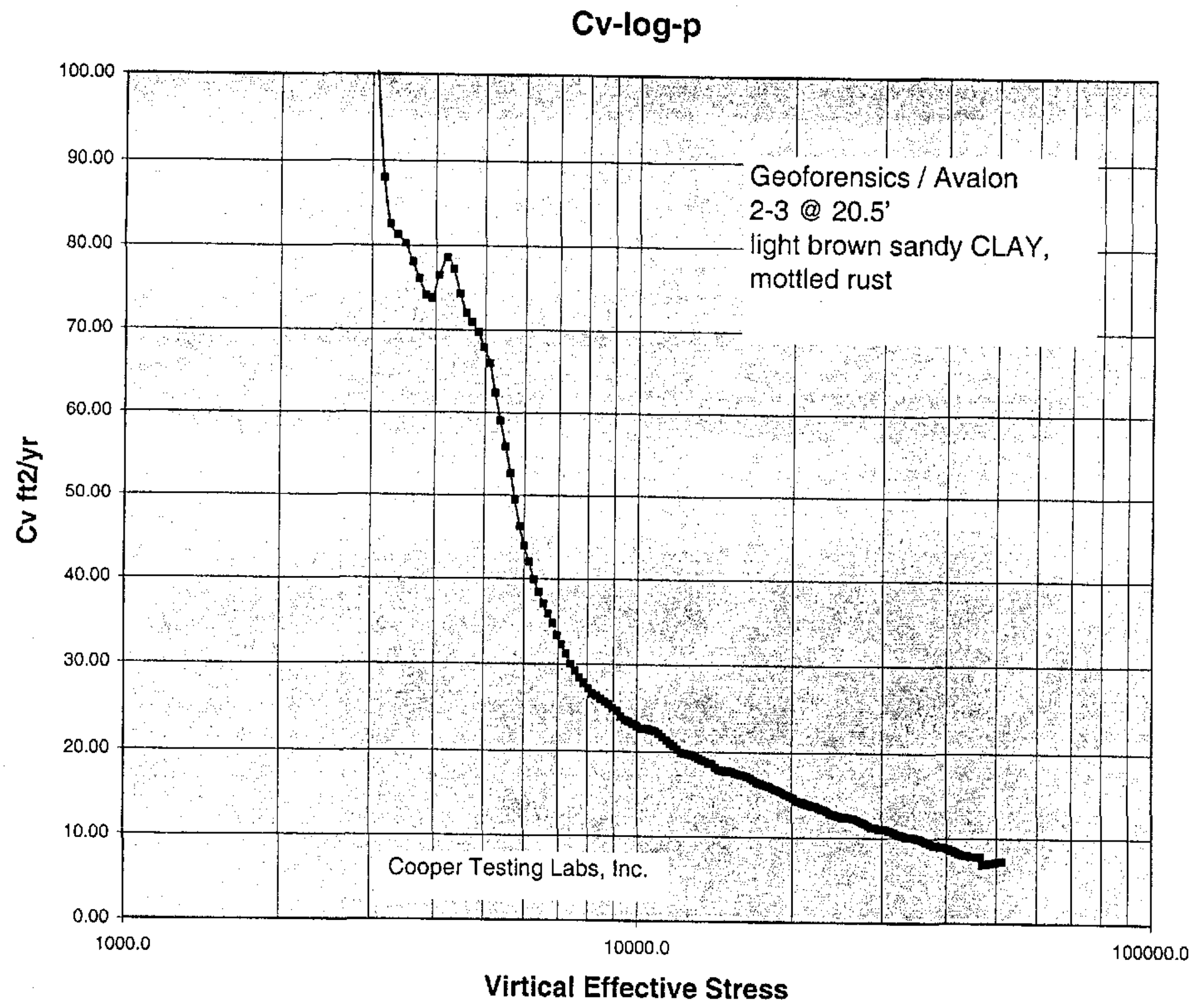
Pressure (ksf)	**Final Dial (in.)**	**Machine Defl. (in.)**	**C_v (ft.^{2}/day)**	**Void Ratio**	**% Compression /Swell**
start	0.00000			0.843	
0.15	0.00520	0.00000	1.51	0.834	0.5 Comprs.
0.30	0.00820	0.00000	0.05	0.828	0.8 Comprs.
0.55	0.01480	0.00000	0.08	0.816	1.5 Comprs.
1.10	0.02780	0.00000	0.08	0.792	2.8 Comprs.
2.20	0.04870	0.00000	0.12	0.753	4.9 Comprs.
4.40	0.08520	0.00000	0.12	0.686	8.5 Comprs.
8.80	0.12650	0.00000	0.05	0.610	12.7 Comprs.
17.60	0.17570	0.00000	0.05	0.519	17.6 Comprs.
35.20	0.22460	0.00000	0.04	0.429	22.5 Comprs.
8.80	0.21230	0.00000		0.452	21.2 Comprs.
2.20	0.19380	0.00000		0.486	19.4 Comprs.
0.55	0.17640	0.00000		0.518	17.6 Comprs.
0.15	0.16520	0.00000		0.539	16.5 Comprs.

Constant Rate of Strain Consolidation Test

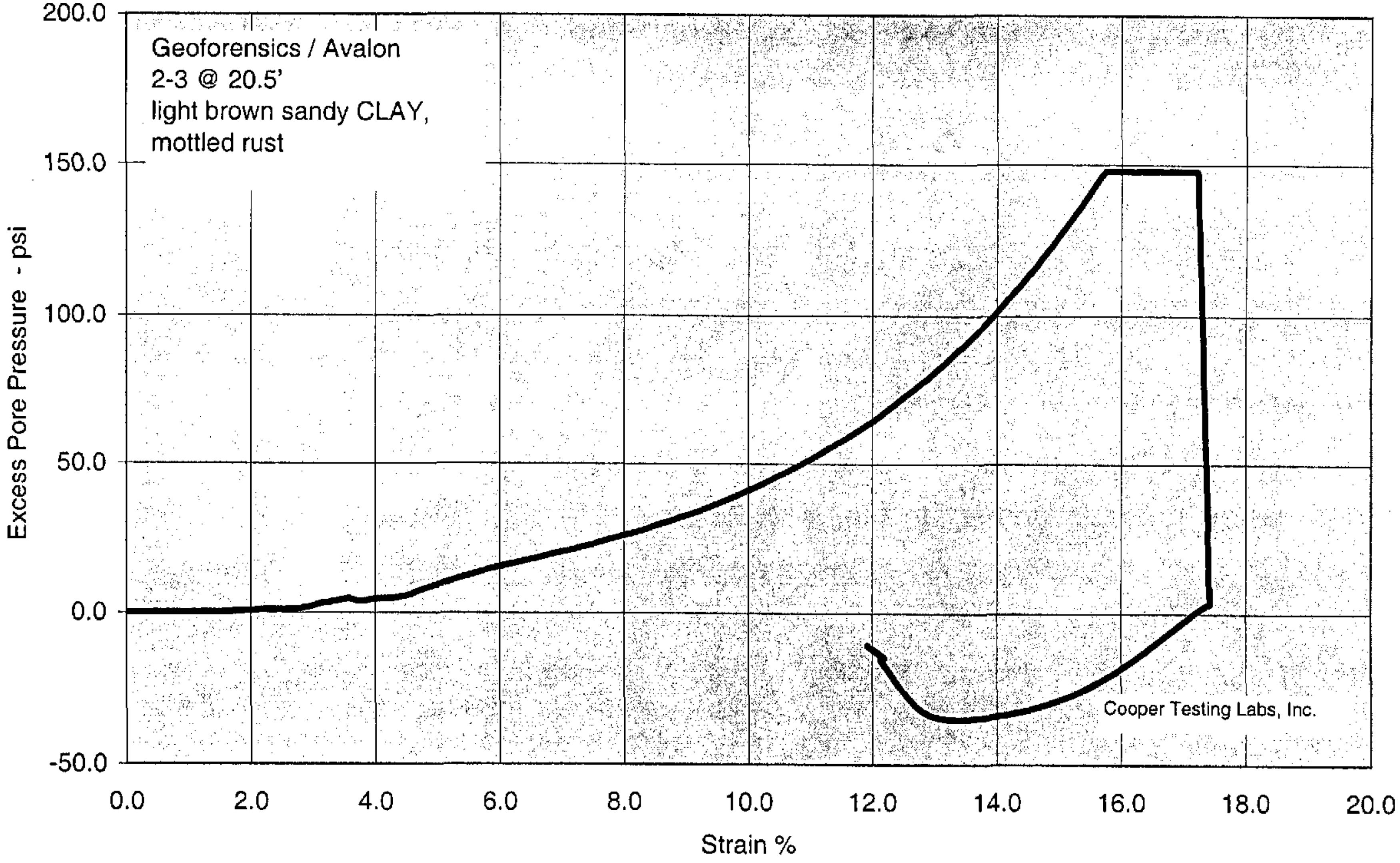
Cooper Testing Labs, Inc.

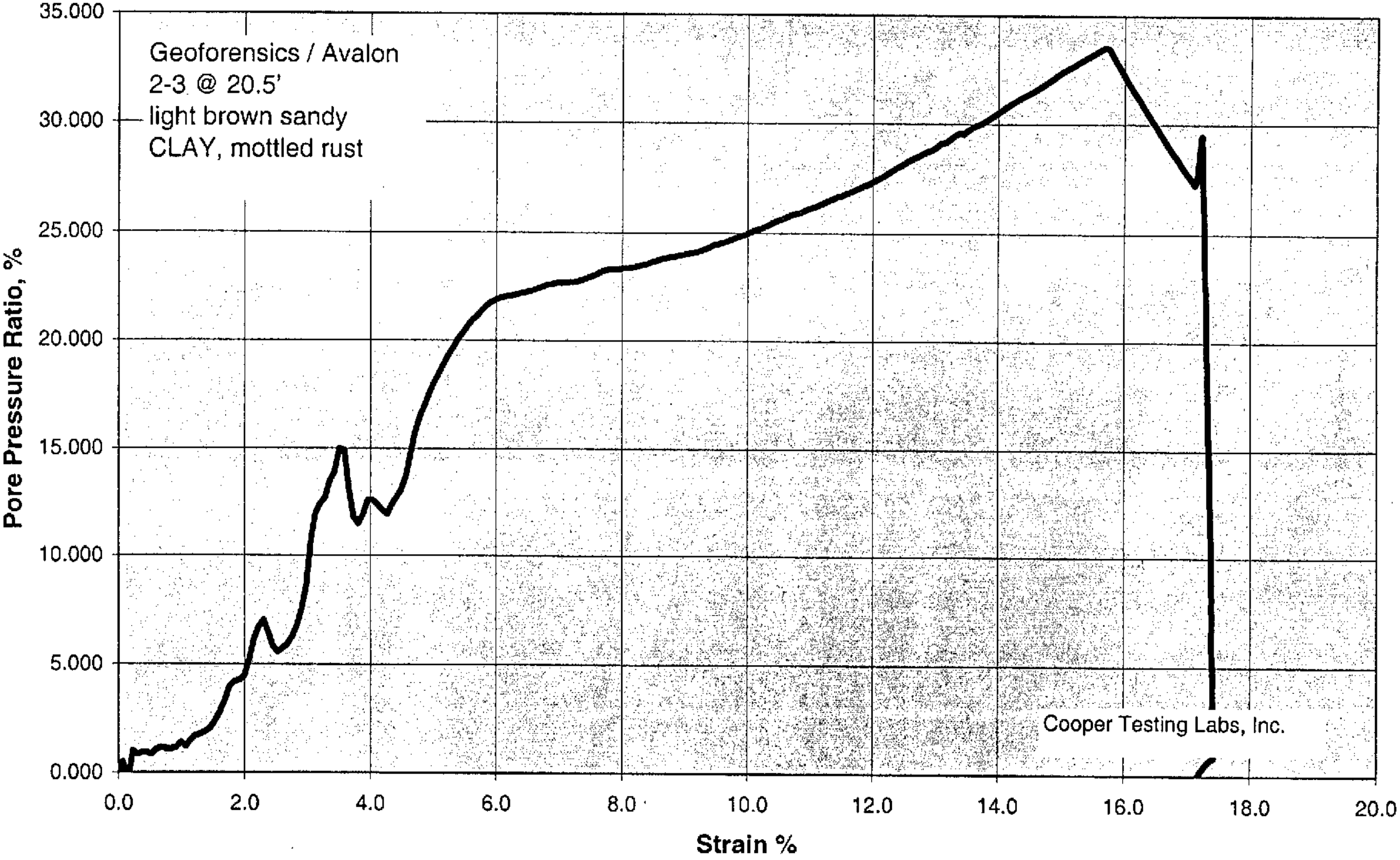
Strain-Log-P Curve




Cv-log-p
100.00
90.00
80.00
70.00
60.00
50.00
40.00
30.00
20.00
10.00
0.00
Cv ft2/yr
Geoforensics / Avalon
2-3 @ 20.5'
light brown sandy CLAY,
mottled rust
Cooper Testing Labs, Inc.
1000.0
10000.0
100000.0
Virtical Effective Stress

Excess Pore Pressure versus Strain

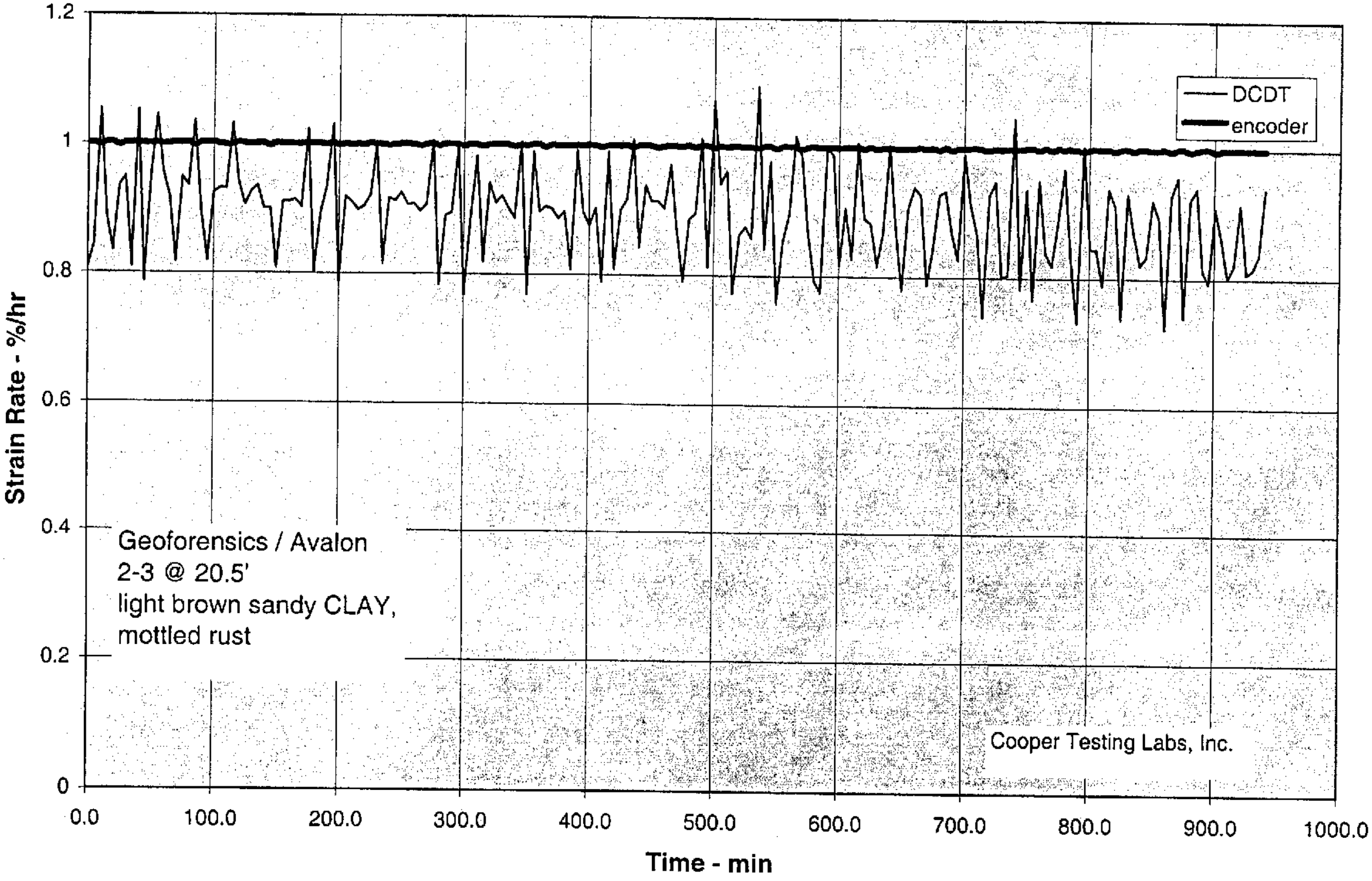




Pore Pressure Ratio

Geoforensics / Avalon
2-3 @ 20.5'
light brown sandy
CLAY, mottled rust

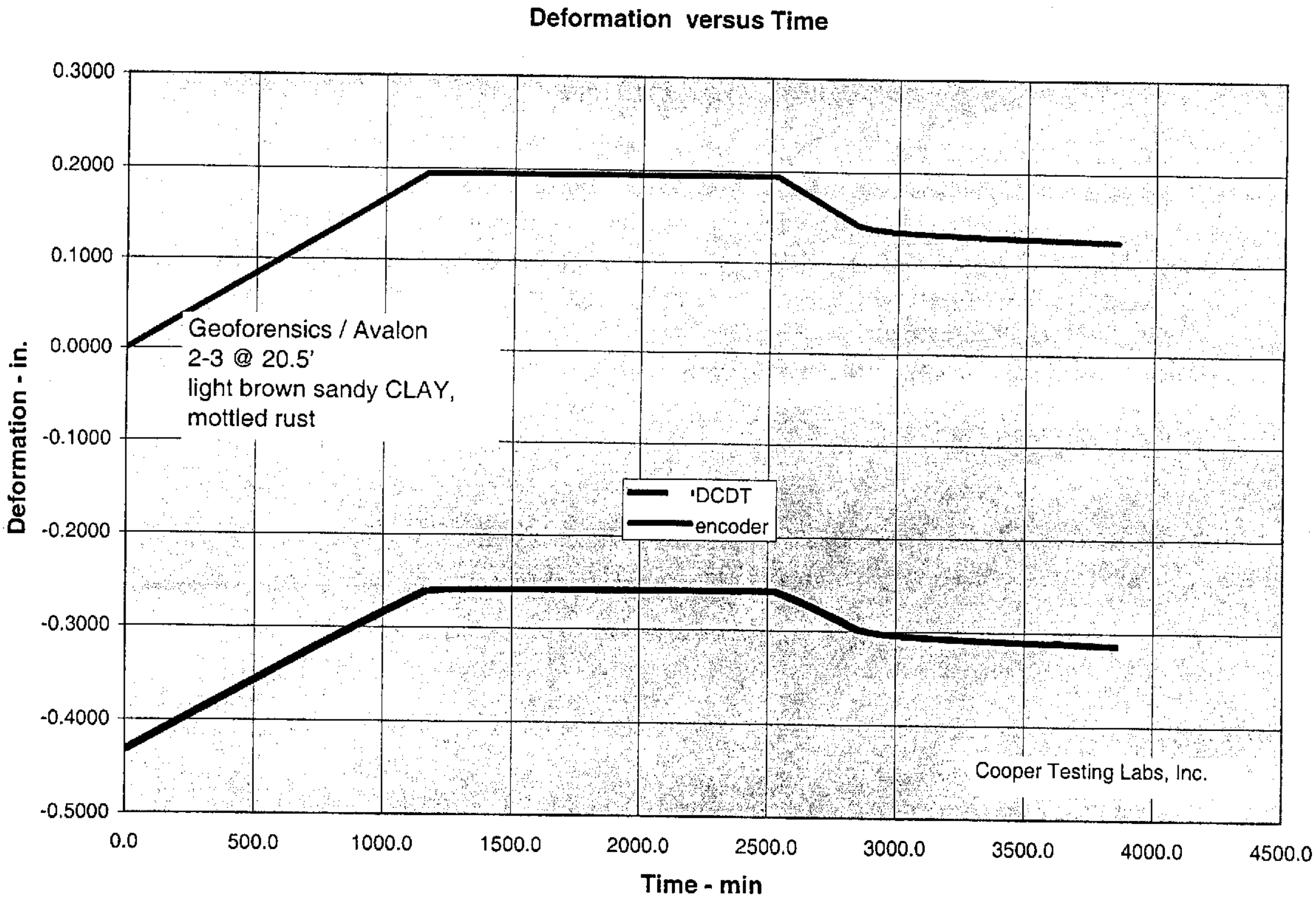
Cooper Testing Labs, Inc.



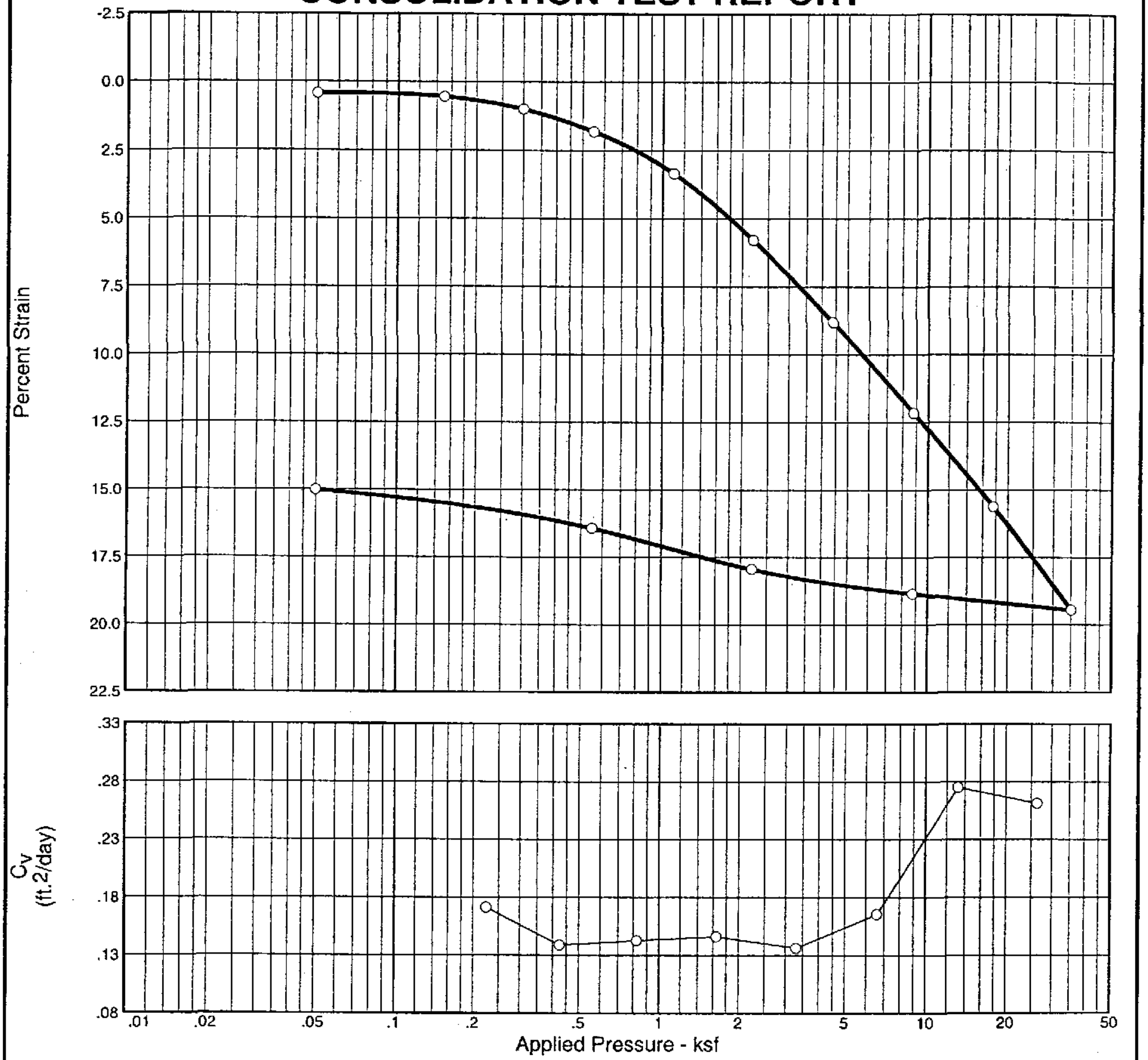
Strain Rate versus Time

Geoforensics / Avalon
2-3 @ 20.5'
light brown sandy CLAY,
mottled rust

Cooper Testing Labs, Inc.


Deformation versus Time
0.3000
0.2000
0.1000
0.0000
-0.1000
-0.2000
-0.3000
-0.4000
-0.5000
Deformation - in.
0.0
500.0
1000.0
1500.0
2000.0
2500.0
3000.0
3500.0
4000.0
4500.0
Time - min
Geoforensics / Avalon
2-3 @ 20.5'
light brown sandy CLAY,
mottled rust
DCDT
encoder
Cooper Testing Labs, Inc.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
89.0 %	26.0 %	94.2			2.7			0.790

MATERIAL DESCRIPTION

gray silty CLAY w/sand

Project No. 060-1192 **Client:** Geoforensics

Project: Avalon

Source: 3-1 **Elev./Depth:** 4.5'

Remarks:

CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

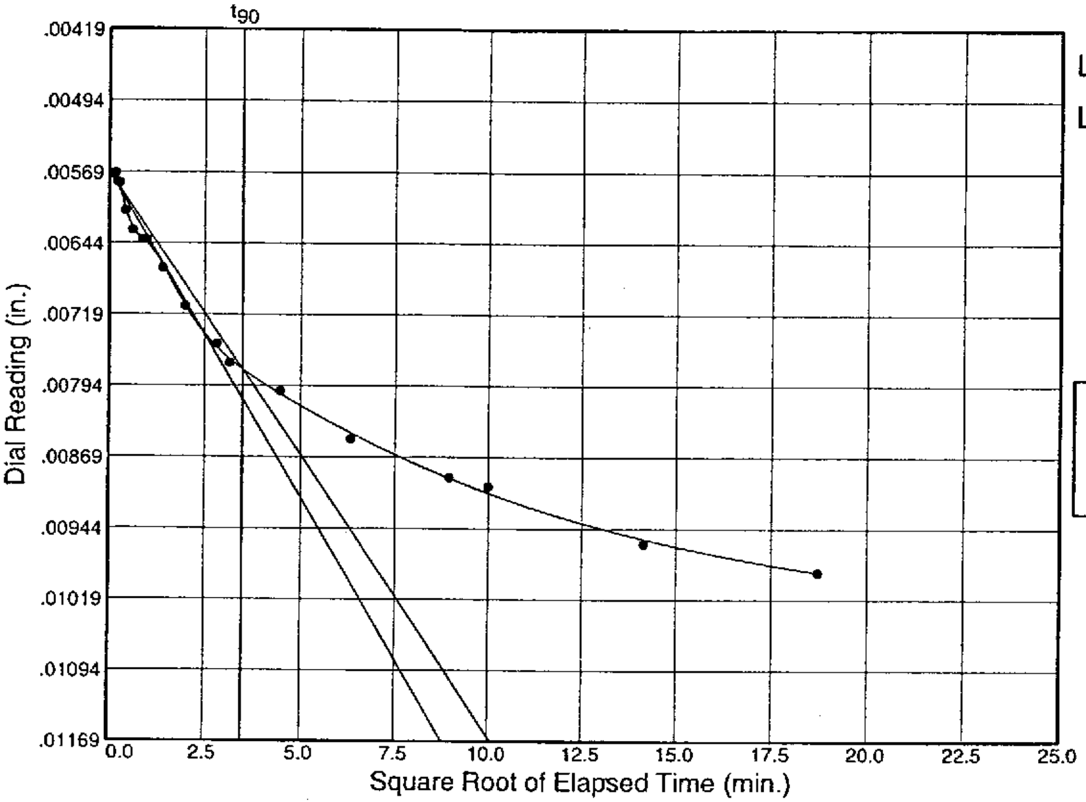
Plate

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-1

Elev./Depth: 4.5'



Load No.= 3

Load= 0.30 ksf

D_0 = 0.00569

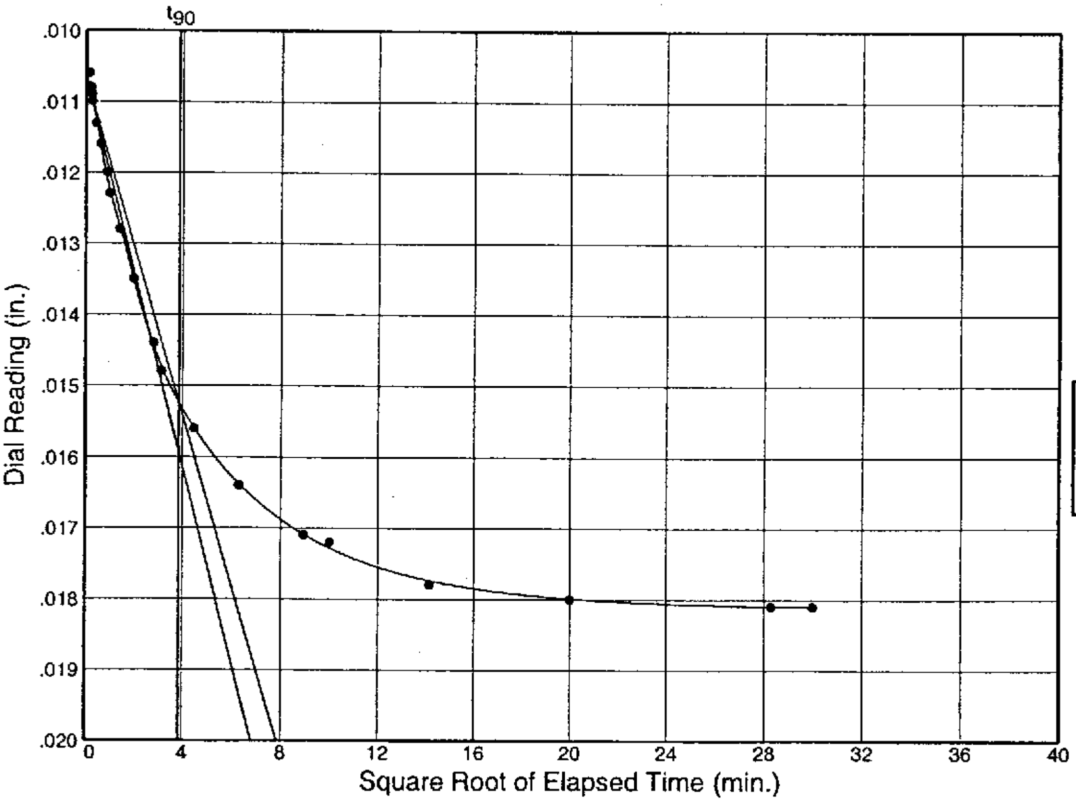
D_{90} = 0.00777

D_{100} = 0.00800

T_{90} = 12.18 min.

C_v @ T_{90}

0.17 ft.^{2}/day



Load No.= 4

Load= 0.55 ksf

D_0 = 0.01065

D_{90} = 0.01524

D_{100} = 0.01575

T_{90} = 14.85 min.

C_v @ T_{90}

0.14 ft.^{2}/day

Plate

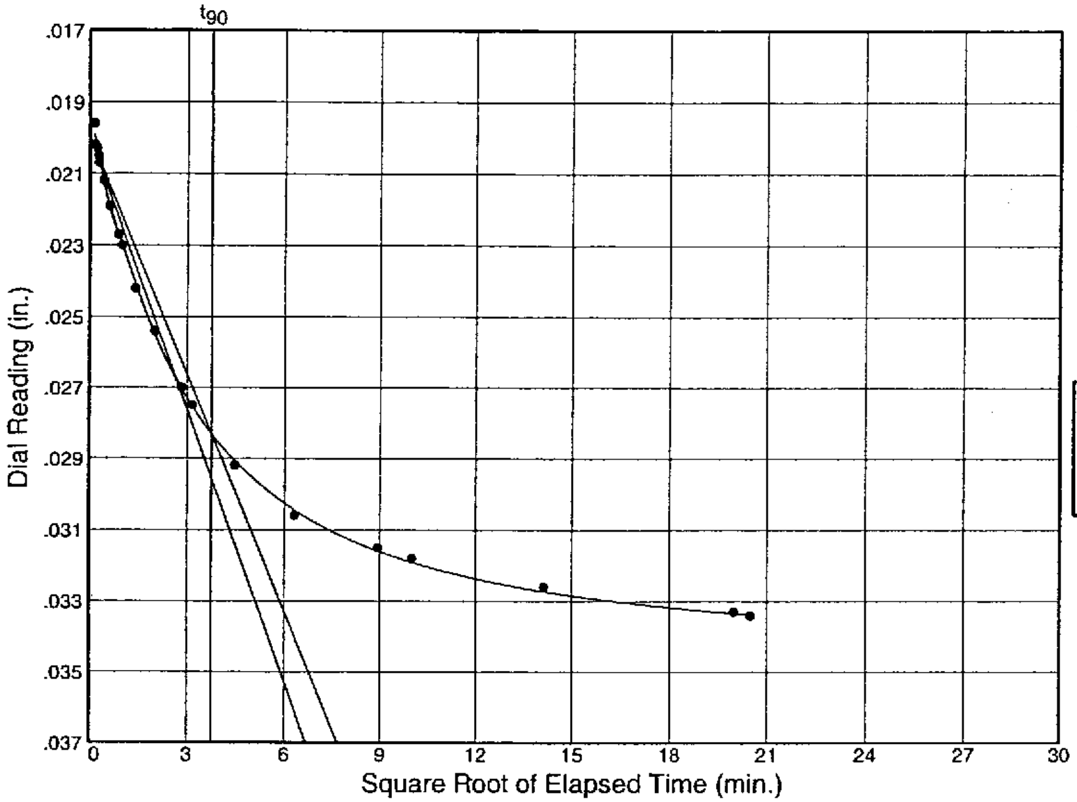
COOPER TESTING LABORATORY

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-1

Elev./Depth: 4.5'



Load No.= 5

Load= 1.10 ksf

D_0 = 0.02001

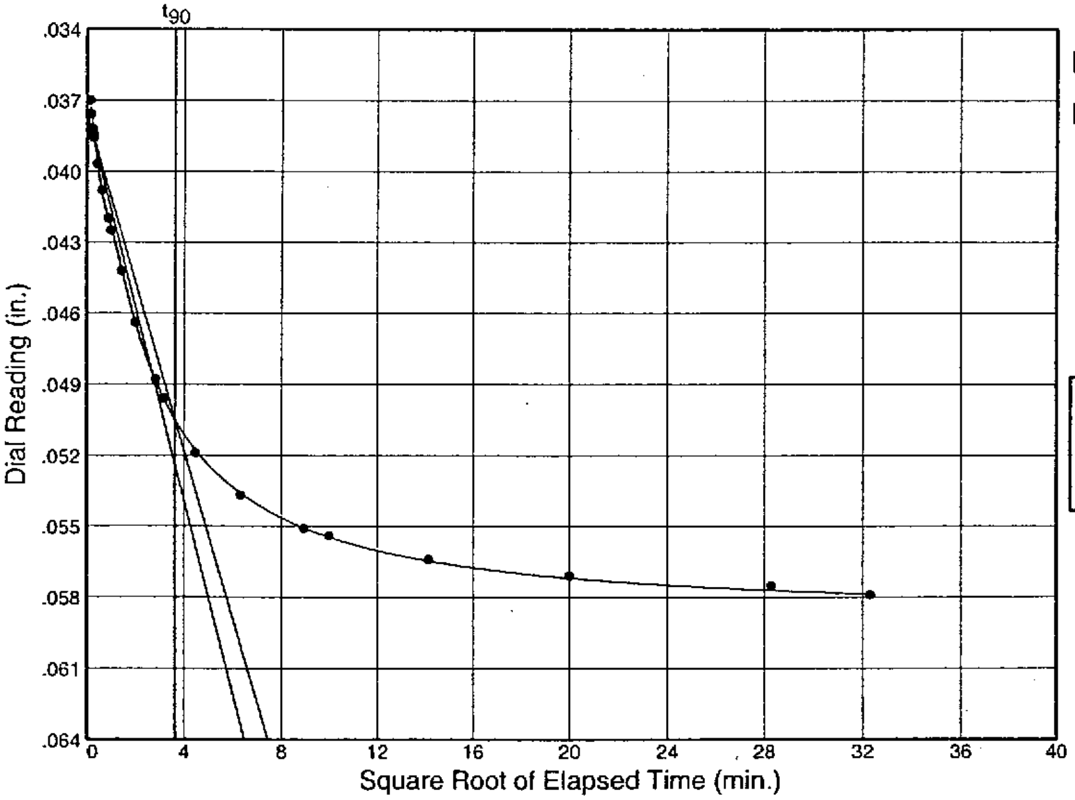
D_{90} = 0.02832

D_{100} = 0.02924

T_{90} = 14.10 min.

C_v @ T_{90}

0.14 ft.^{2}/day



Load No.= 6

Load= 2.20 ksf

D_0 = 0.03763

D_{90} = 0.05053

D_{100} = 0.05197

T_{90} = 13.20 min.

C_v @ T_{90}

0.15 ft.^{2}/day

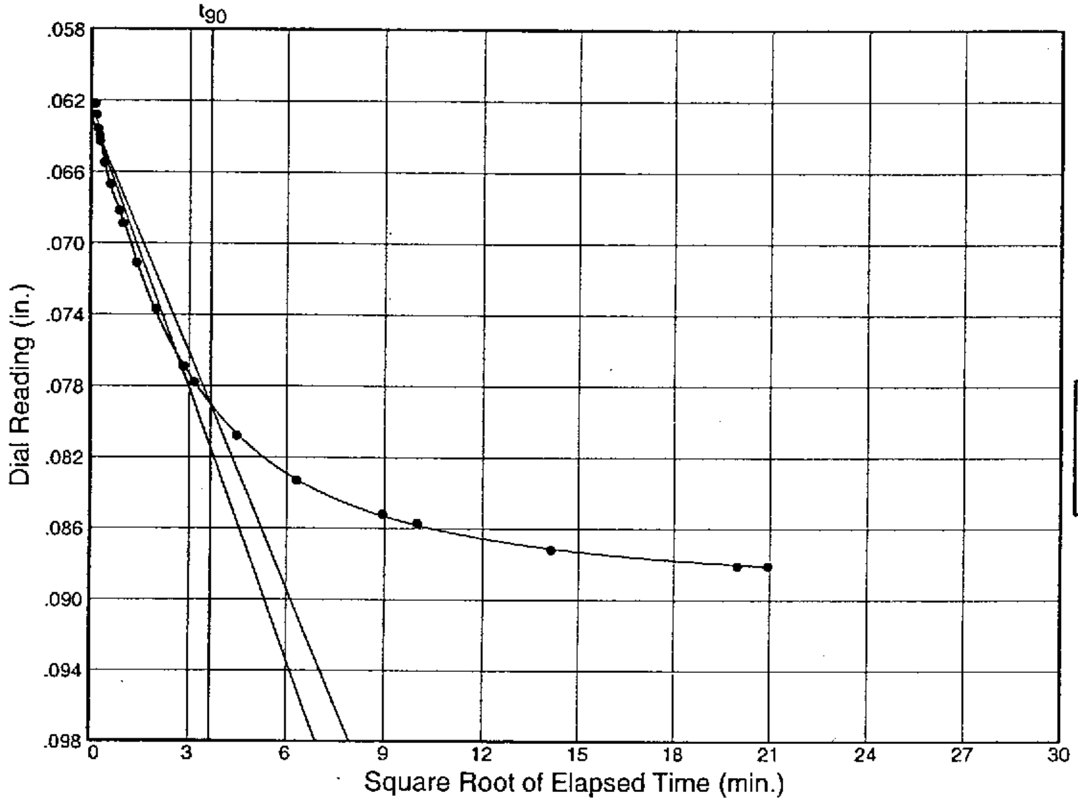
Plate

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-1

Elev./Depth: 4.5'



Load No.= 7

Load= 4.40 ksf

D_0 = 0.06291

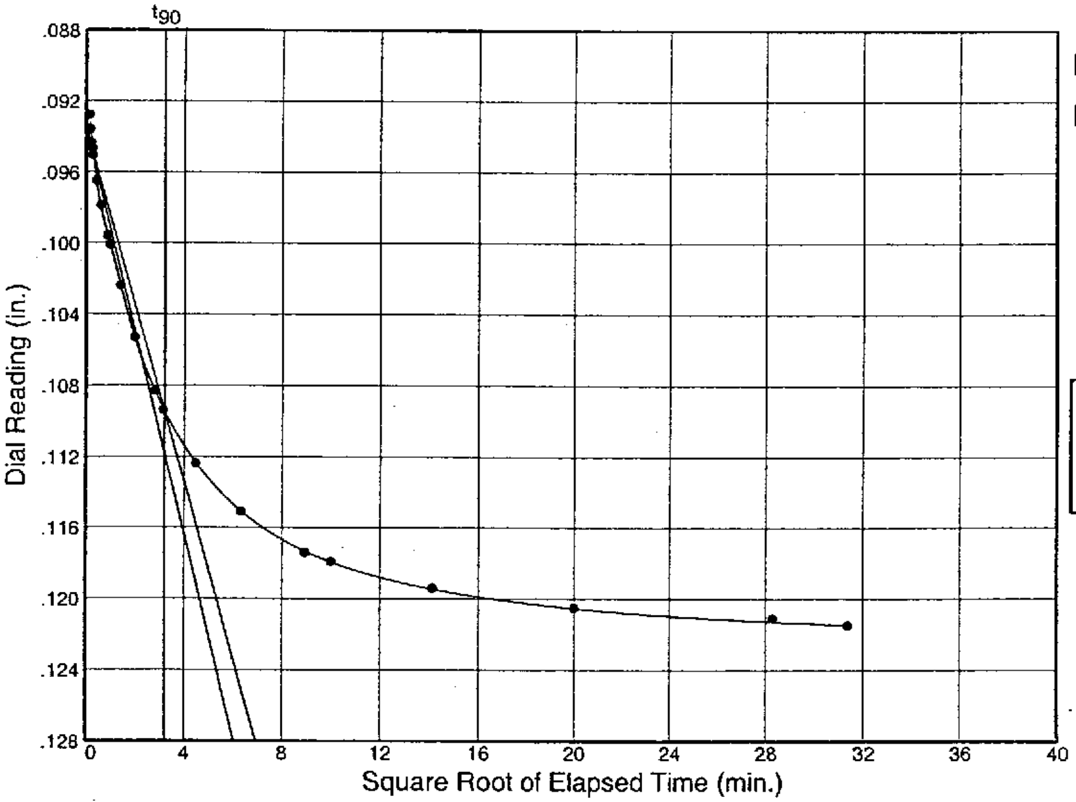
D_{90} = 0.07904

D_{100} = 0.08083

T_{90} = 13.37 min.

C_V @ T_{90}

0.14 ft.^{2}/day



Load No.= 8

Load= 8.80 ksf

D_0 = 0.09351

D_{90} = 0.10947

D_{100} = 0.11124

T_{90} = 10.27 min.

C_V @ T_{90}

0.17 ft.^{2}/day

Plate

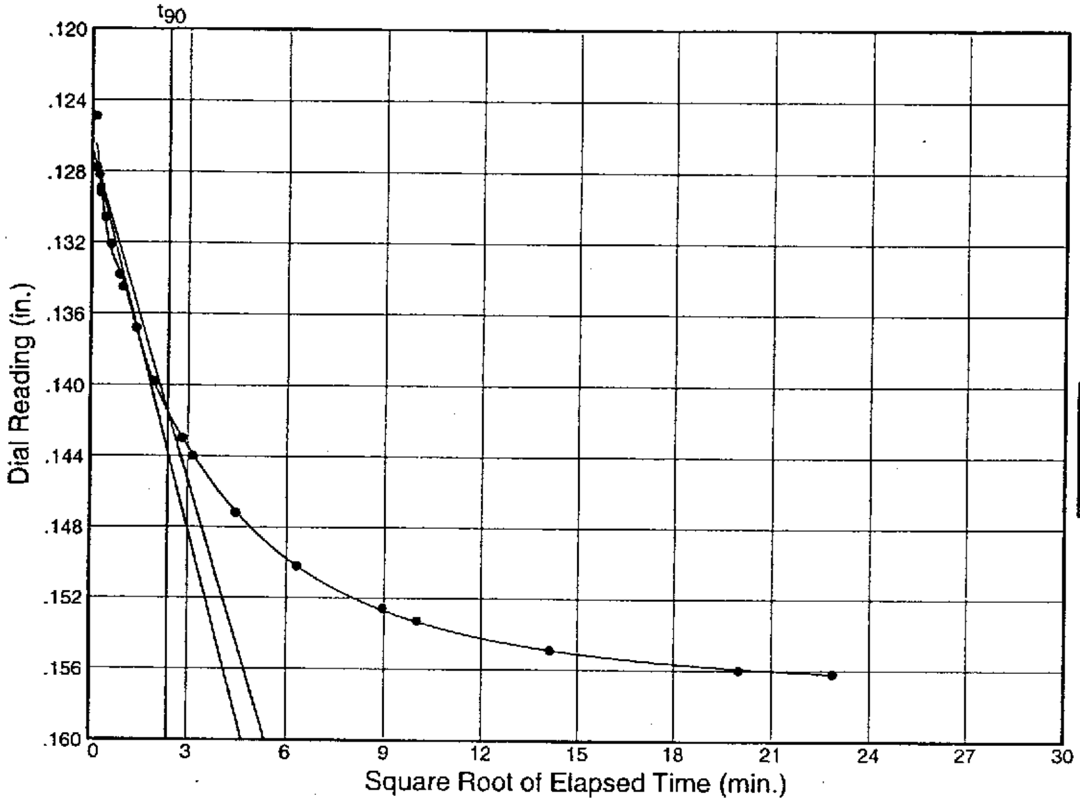
COOPER TESTING LABORATORY

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-1

Elev./Depth: 4.5'



Load No.= 9

Load= 17.60 ksf

D_0 = 0.12672

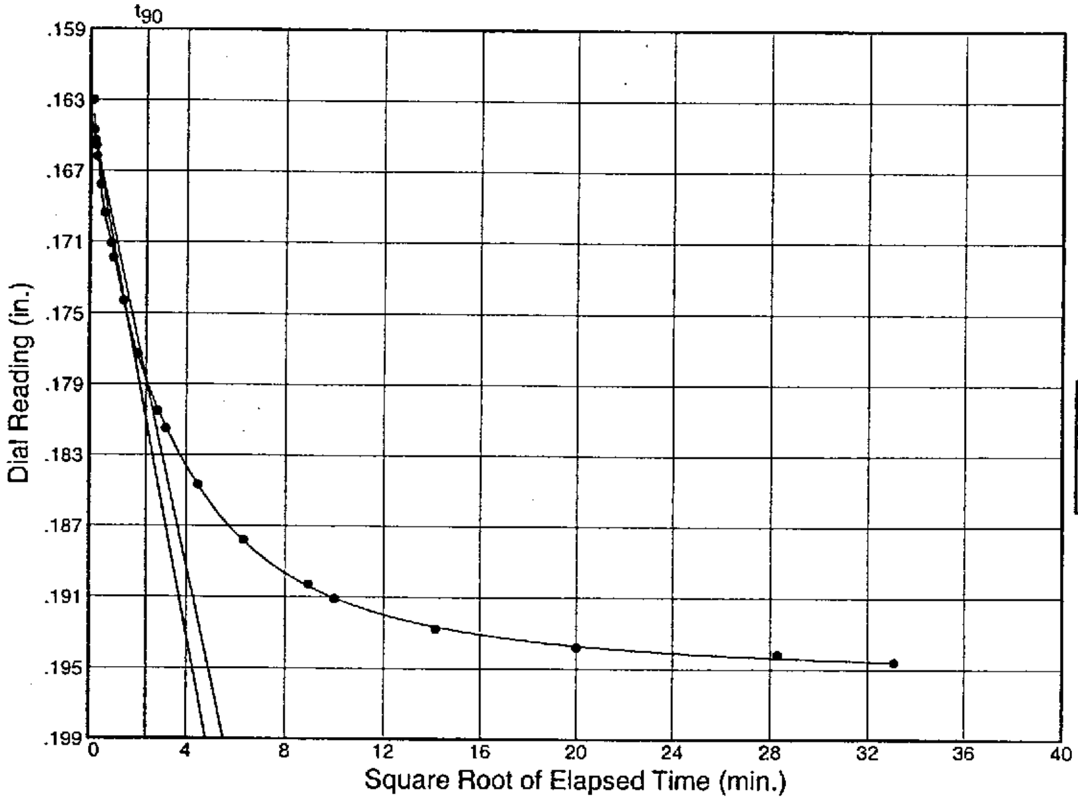
D_{90} = 0.14152

D_{100} = 0.14317

T_{90} = 5.72 min.

C_V @ T_{90}

0.28 ft.^{2}/day



Load No.= 10

Load= 35.20 ksf

D_0 = 0.16386

D_{90} = 0.17881

D_{100} = 0.18047

T_{90} = 5.52 min.

C_V @ T_{90}

0.26 ft.^{2}/day

Plate

CONSOLIDATION TEST DATA

Client: Geoforensics
Project: Avalon
Project Number: 060-1192

Sample Data

Source: 3-1
Sample No.:
Elev. or Depth: 4.5′ **Sample Length (in./cm.):**
Location:
Description: gray silty CLAY w/sand
Liquid Limit: **Plasticity Index:**
USCS: **AASHTO:** **Figure No.:**
Testing Remarks:

Test Specimen Data

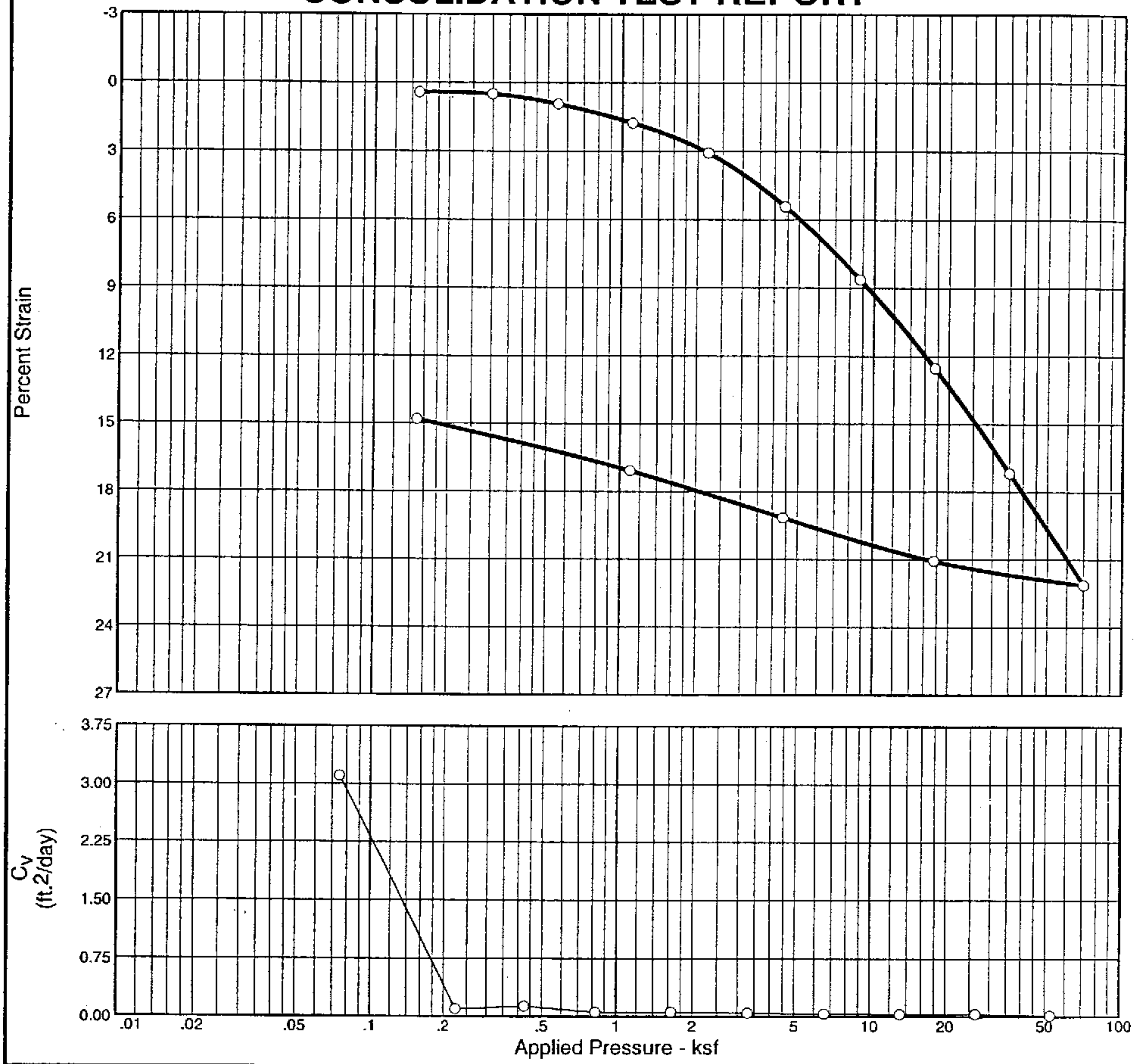
TOTAL SAMPLE	BEFORE TEST	AFTER TEST
Wet w+t = 143.30 g.	**Consolidometer #** = 1	**Wet w+t** = 133.90 g.
Dry w+t = 113.70 g.		**Dry w+t** = 113.70 g.
Tare Wt. = .00 g.	**Spec. Gravity** = 2.7	**Tare Wt.** = .00 g.
Height = 1.00 in.	**Height** = 1.00 in.	
Diameter = 2.42 in.	**Diameter** = 2.42 in.	
Weight = 143.30 g.	**Defl. Table** = n/a	
Moisture = 26.0 %	**Ht. Solids** = 0.5587 in.	**Moisture** = 17.8 %
Wet Den. = 118.7 pcf	**Dry Wt.** = 113.70 g.*	**Dry Wt.** = 113.70 g.
Dry Den. = 94.2 pcf	**Void Ratio** = 0.790	**Void Ratio** = 0.521
	Saturation = 89.0 %	

***Initial dry weight used in calculations**

End-of-Load Summary

Pressure (ksf)	Final Dial (in.)	Machine Defl. (in.)	C_v (ft.^{2}/day)	Void Ratio	% Compression /Swell
start	0.00000			0.790	
0.05	0.00420	0.00000		0.782	0.4 Comprs.
0.15	0.00540	0.00000		0.780	0.5 Comprs.
0.30	0.00990	0.00000	0.17	0.772	1.0 Comprs.
0.55	0.01810	0.00000	0.14	0.757	1.8 Comprs.
1.10	0.03340	0.00000	0.14	0.730	3.3 Comprs.
2.20	0.05790	0.00000	0.15	0.686	5.8 Comprs.
4.40	0.08810	0.00000	0.14	0.632	8.8 Comprs.
8.80	0.12150	0.00000	0.17	0.572	12.2 Comprs.
17.60	0.15620	0.00000	0.28	0.510	15.6 Comprs.
35.20	0.19460	0.00000	0.26	0.442	19.5 Comprs.
8.80	0.18850	0.00000		0.452	18.9 Comprs.
2.20	0.17940	0.00000		0.469	17.9 Comprs.
0.55	0.16430	0.00000		0.496	16.4 Comprs.
0.05	0.15020	0.00000		0.521	15.0 Comprs.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
99.7 %	24.7 %	101.0			2.7			0.668

MATERIAL DESCRIPTION

dark gray sandy CLAY w/fine gravel

Project No. 060-1192 **Client:** Geoforensics

Project: Avalon

Source: 3-2 **Elev./Depth:** 9.5'

CONSOLIDATION TEST REPORT

COOPER TESTING LABORATORY

Remarks:

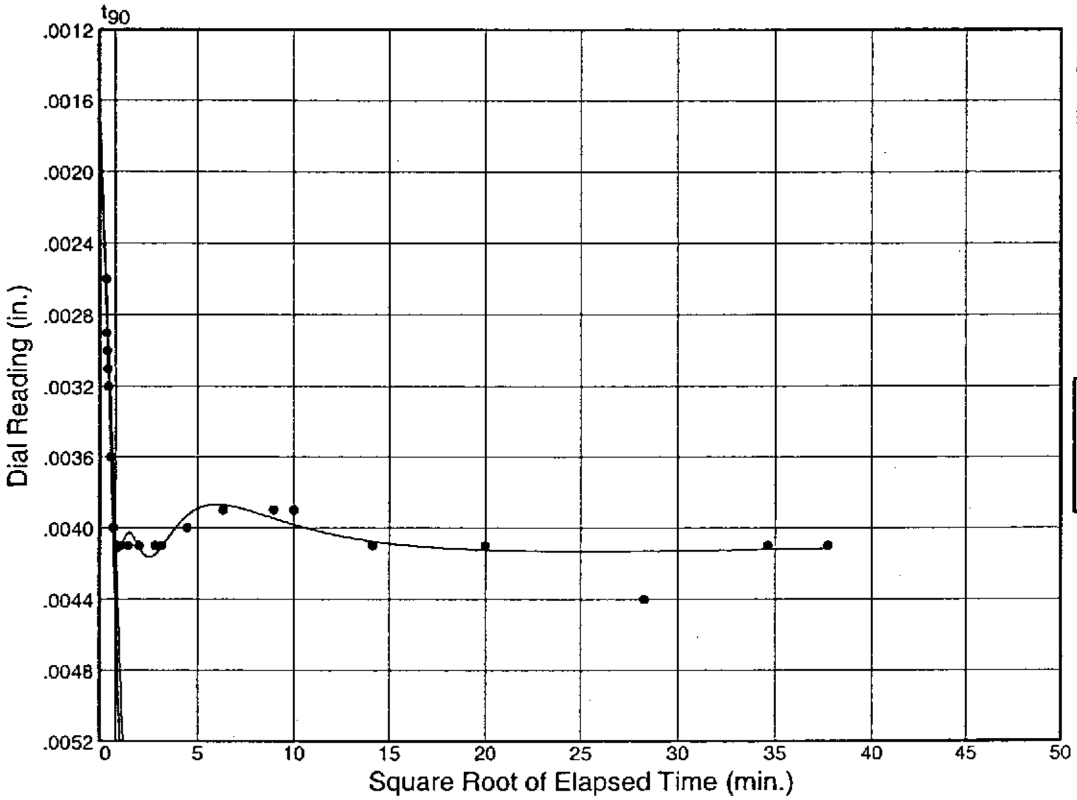
Plate

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-2

Elev./Depth: 9.5'



Load No.= 1

Load= 0.15 ksf

D_0 = 0.00161

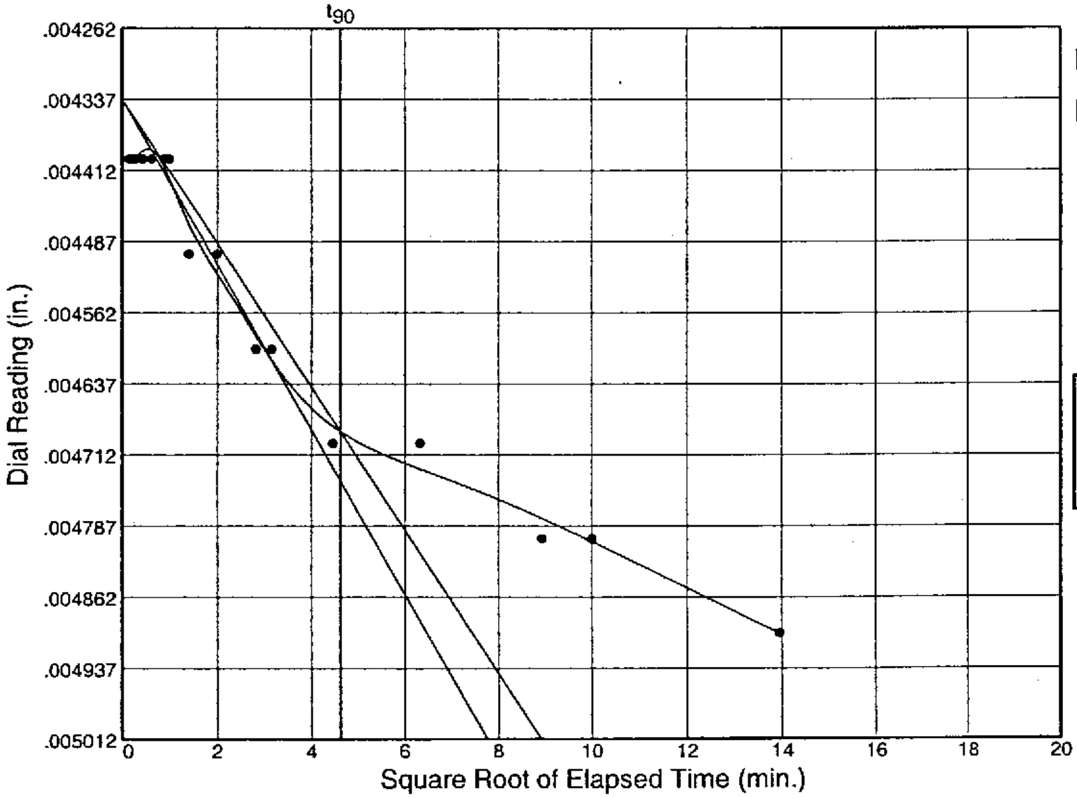
D_{90} = 0.00412

D_{100} = 0.00440

T_{90} = 0.68 min.

C_V @ T_{90}

3.11 ft.^{2}/day



Load No.= 2

Load= 0.30 ksf

D_0 = 0.00434

D_{90} = 0.00469

D_{100} = 0.00473

T_{90} = 21.42 min.

C_V @ T_{90}

0.10 ft.^{2}/day

Plate

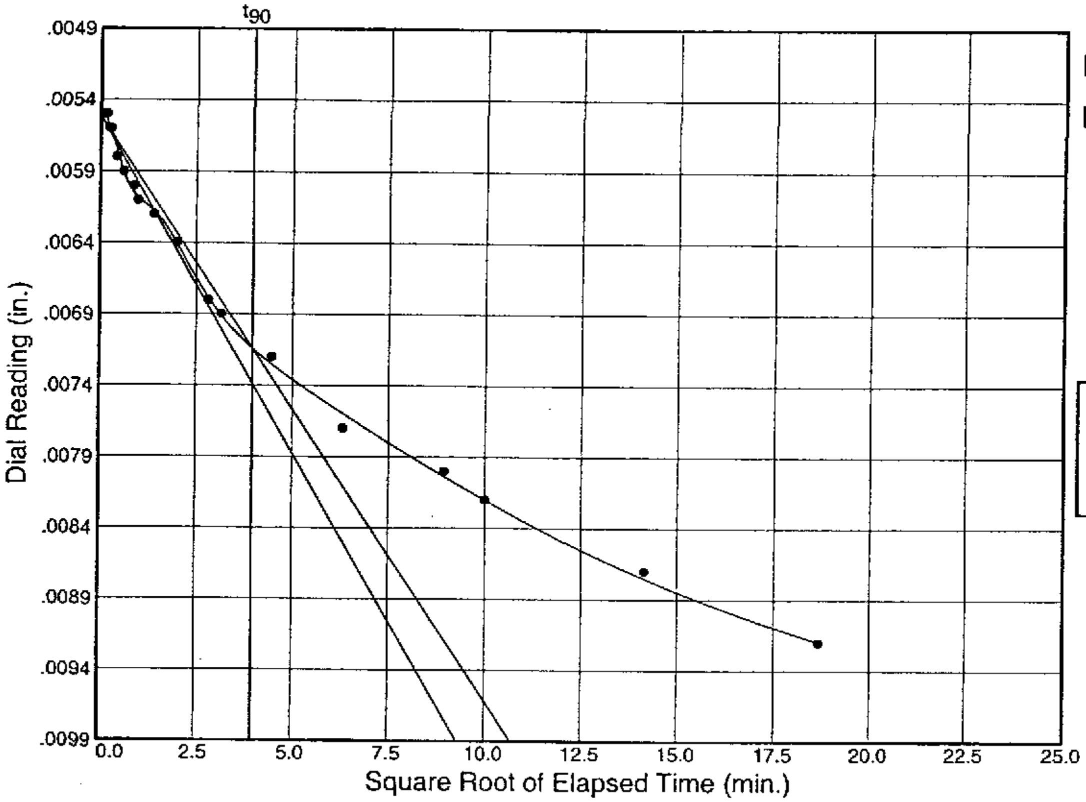
COOPER TESTING LABORATORY

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-2

Elev./Depth: 9.5'



Load No.= 3

Load= 0.55 ksf

D_0 = 0.00551

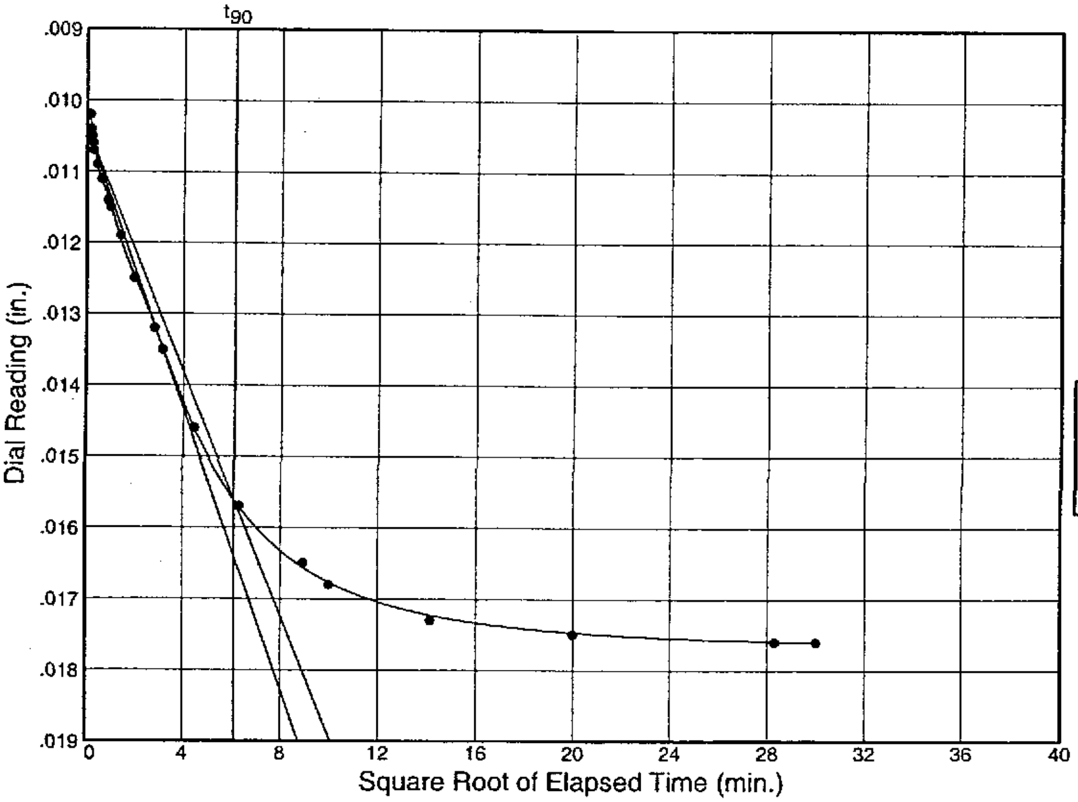
D_{90} = 0.00714

D_{100} = 0.00732

T_{90} = 15.75 min.

C_v @ T_{90}

0.13 ft.^{2}/day



Load No.= 4

Load= 1.10 ksf

D_0 = 0.01039

D_{90} = 0.01563

D_{100} = 0.01621

T_{90} = 37.43 min.

C_v @ T_{90}

0.06 ft.^{2}/day

Plate

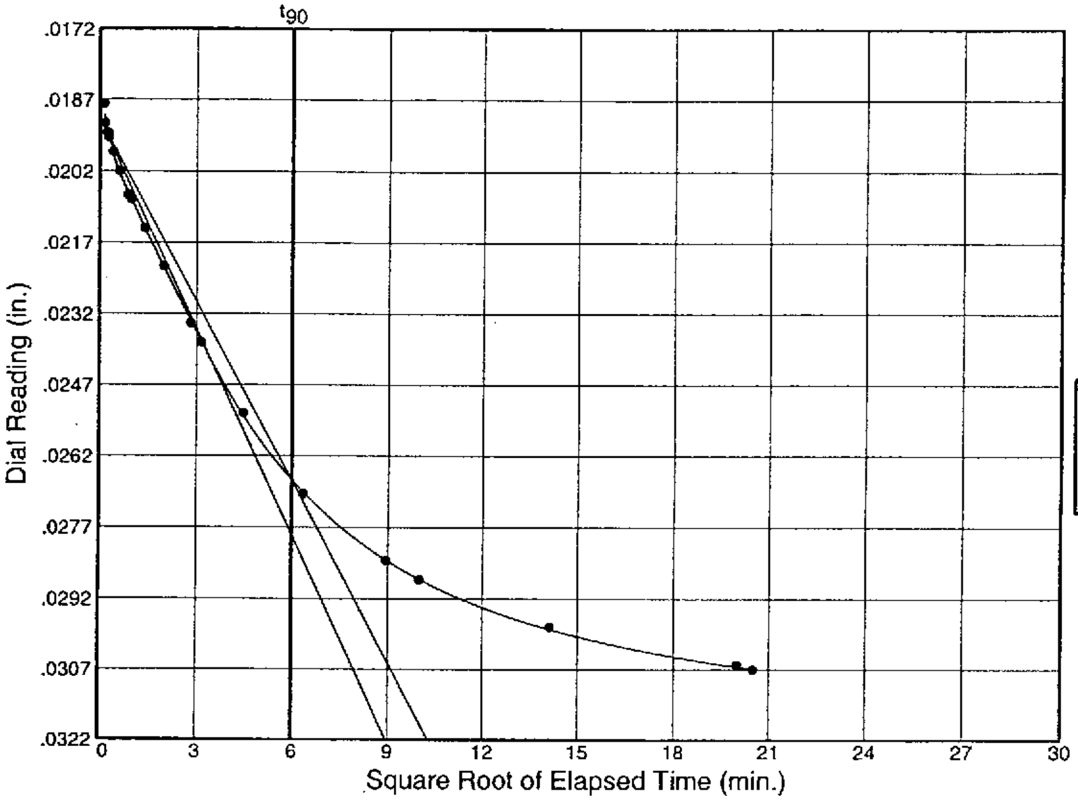
COOPER TESTING LABORATORY

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-2

Elev./Depth: 9.5'



Load No.= 5

Load= 2.20 ksf

D_0 = 0.01913

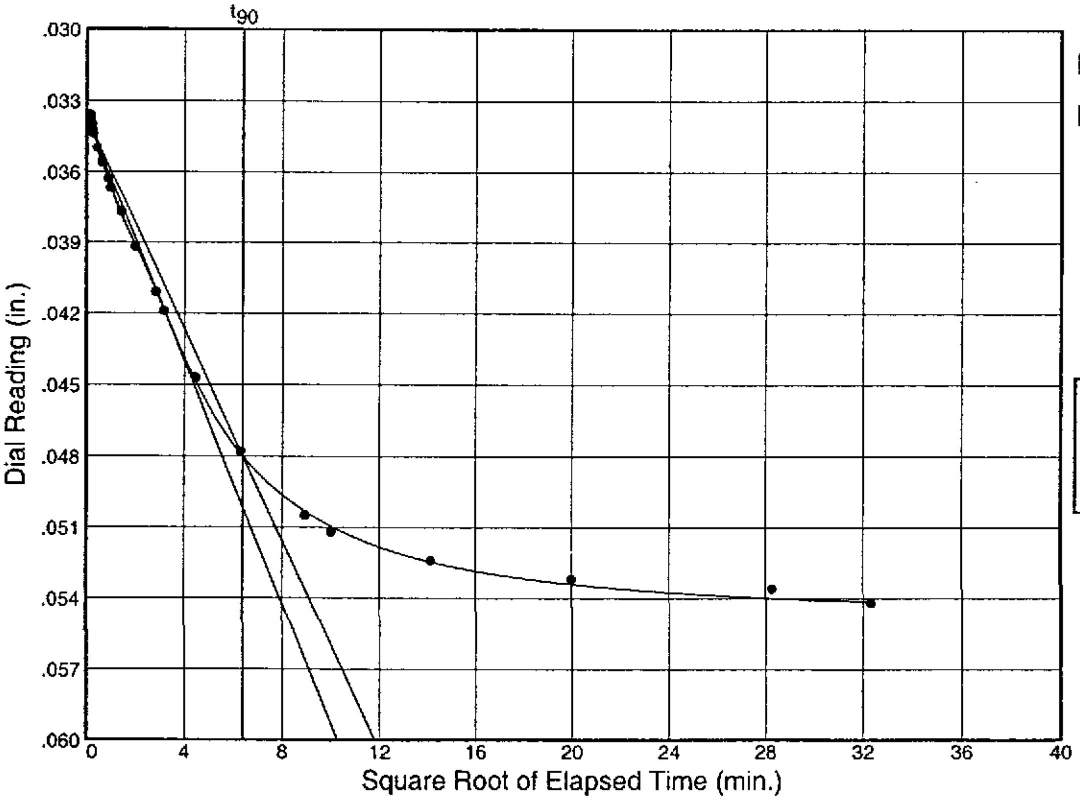
D_{90} = 0.02668

D_{100} = 0.02752

T_{90} = 35.33 min.

C_v @ T_{90}

0.06 ft.^{2}/day



Load No.= 6

Load= 4.40 ksf

D_0 = 0.03376

D_{90} = 0.04799

D_{100} = 0.04957

T_{90} = 40.86 min.

C_v @ T_{90}

0.05 ft.^{2}/day

Plate

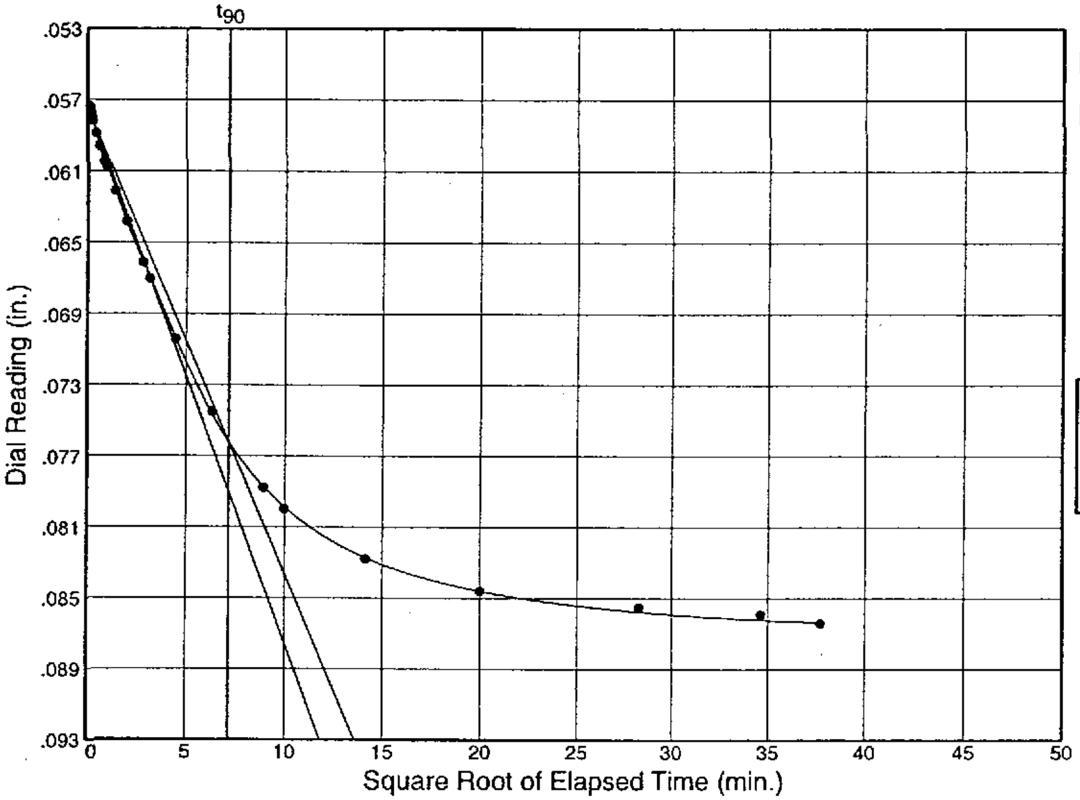
COOPER TESTING LABORATORY

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-2

Elev./Depth: 9.5'



Load No.= 7

Load= 8.80 ksf

D_0 = 0.05747

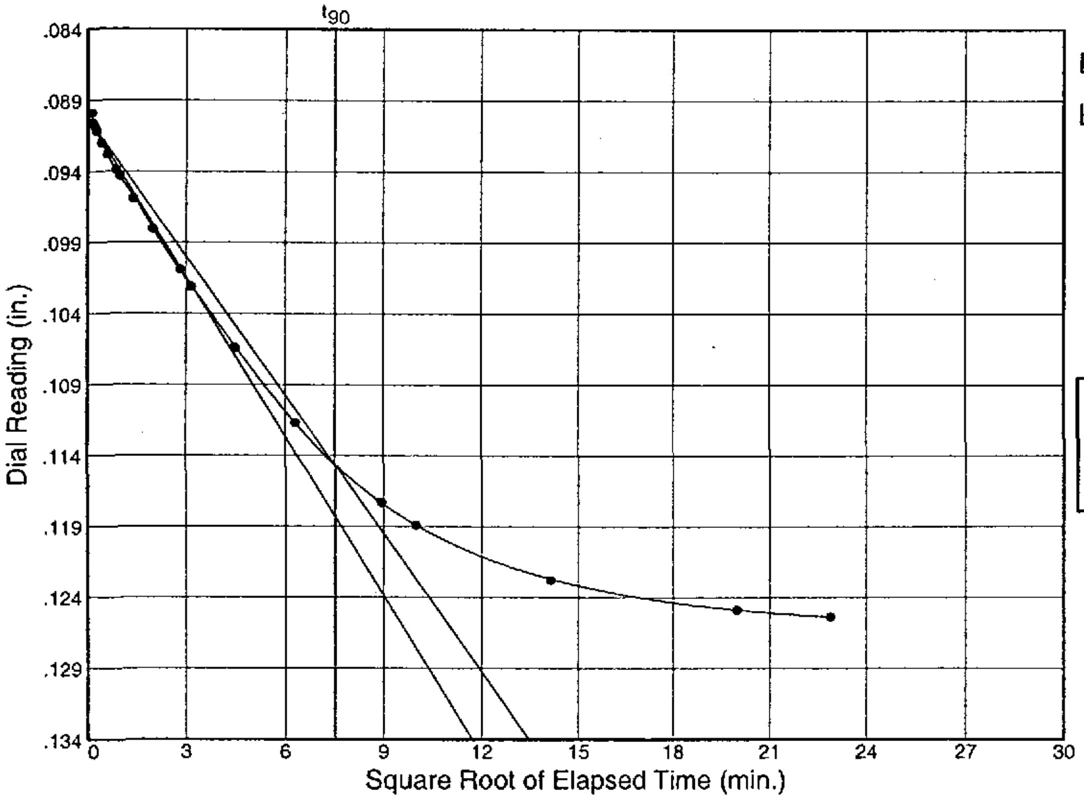
D_{90} = 0.07623

D_{100} = 0.07831

T_{90} = 51.45 min.

C_v @ T_{90}

0.04 ft.^{2}/day



Load No.= 8

Load= 17.60 ksf

D_0 = 0.09024

D_{90} = 0.11464

D_{100} = 0.11735

T_{90} = 56.46 min.

C_v @ T_{90}

0.03 ft.^{2}/day

Plate

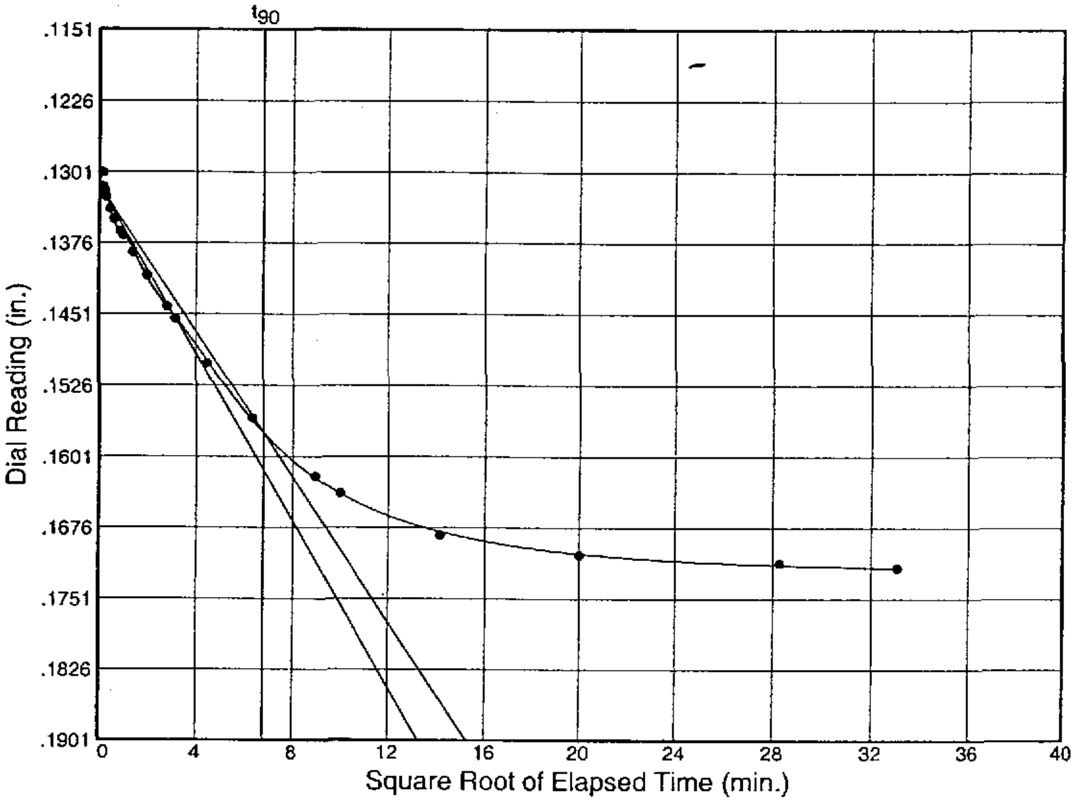
COOPER TESTING LABORATORY

Dial Reading vs. Time

Project No.: 060-1192
Project: Avalon

Source: 3-2

Elev./Depth: 9.5'



Load No.= 9

Load= 35.20 ksf

D_0 = 0.13163

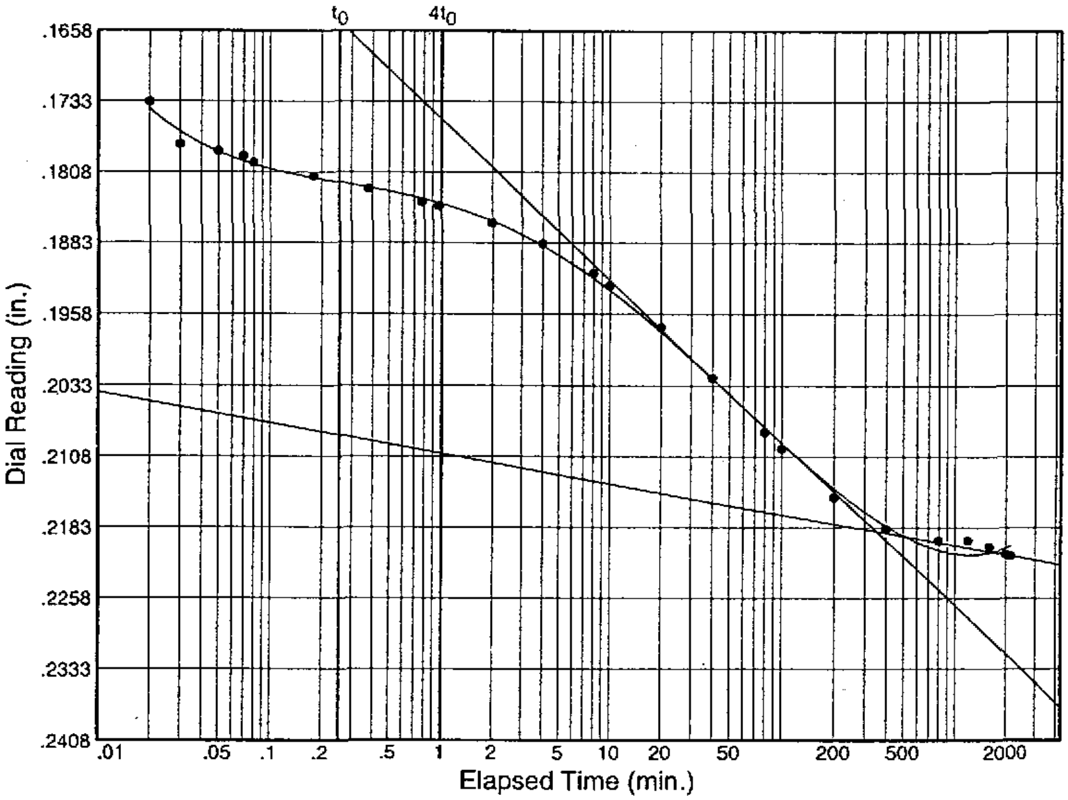
D_{90} = 0.15763

D_{100} = 0.16052

T_{90} = 45.97 min.

C_V @ T_{90}

0.03 ft.^{2}/day



Load No.= 10

Load= 70.40 ksf

D_0 = 0.17966

D_{50} = 0.19920

D_{100} = 0.21875

T_{50} = 25.00 min.

C_V @ T_{50}

0.01 ft.^{2}/day

Plate

COOPER TESTING LABORATORY

CONSOLIDATION TEST DATA

Client: Geoforensics
Project: Avalon
Project Number: 060-1192

Sample Data

Source: 3-2
Sample No.:
Elev. or Depth: 9.5′ **Sample Length (in./cm.):**
Location:
Description: dark gray sandy CLAY w/fine gravel
Liquid Limit: **Plasticity Index:**
USCS: **AASHTO:** **Figure No.:**
Testing Remarks:

Test Specimen Data

TOTAL SAMPLE		**BEFORE TEST**		**AFTER TEST**	
Wet w+t =	152.10 g.	**Consolidometer # =**	1	**Wet w+t =**	146.20 g.
Dry w+t =	122.00 g.			**Dry w+t =**	122.20 g.
Tare Wt. =	.00 g.	**Spec. Gravity =**	2.7	**Tare Wt. =**	.00 g.
Height =	1.00 in.	**Height =**	1.00 in.		
Diameter =	2.42 in.	**Diameter =**	2.42 in.		
Weight =	152.10 g.	**Defl. Table =**	n/a		
Moisture =	24.7 %	**Ht. Solids =**	0.5995 in.	**Moisture =**	19.6 %
Wet Den. =	126.0 pcf	**Dry Wt. =**	122.00 g.*	**Dry Wt. =**	122.20 g.
Dry Den. =	101.0 pcf	**Void Ratio =**	0.668	**Void Ratio =**	0.421
		Saturation =	99.7 %		

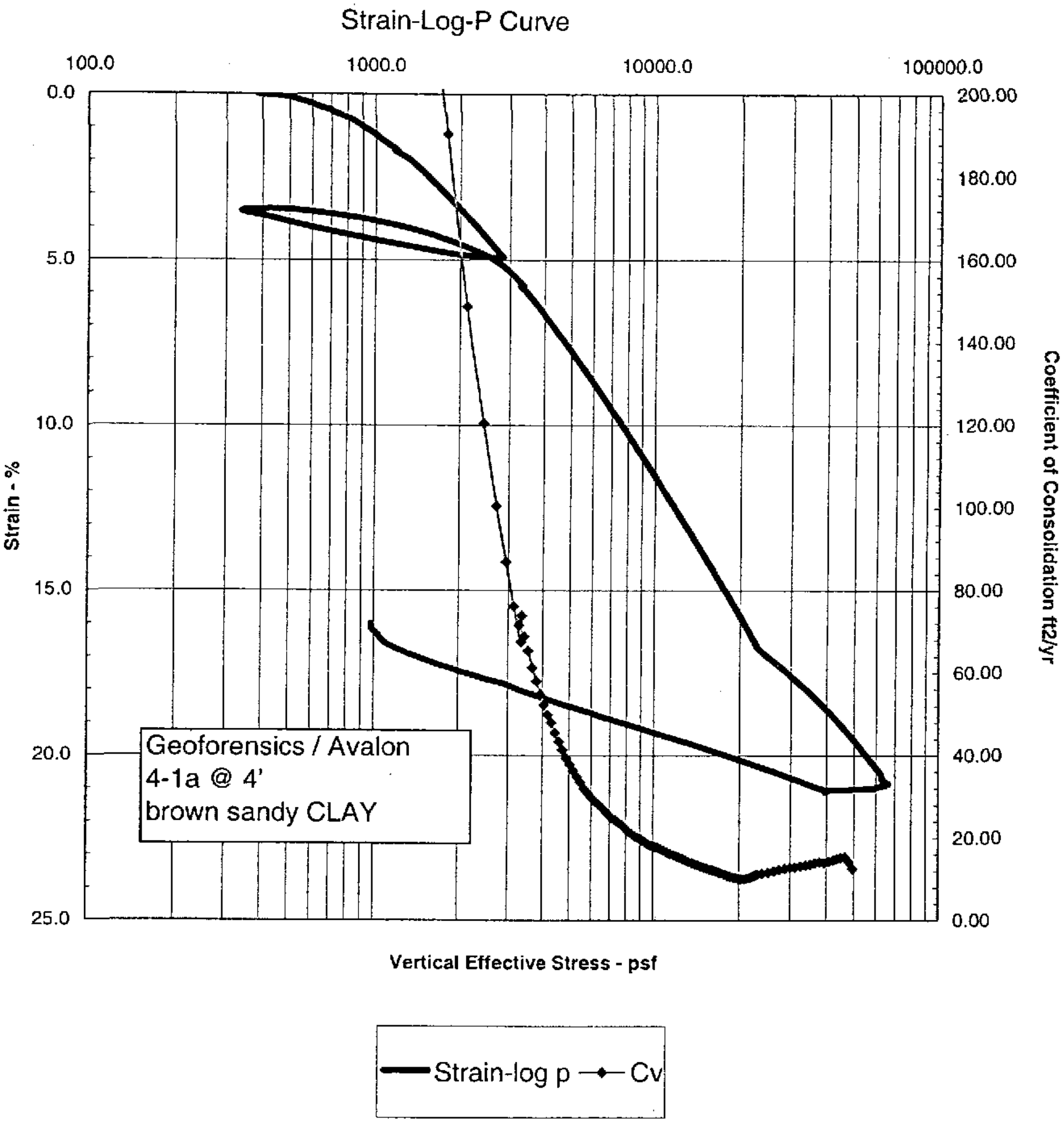
*** Initial dry weight used in calculations**

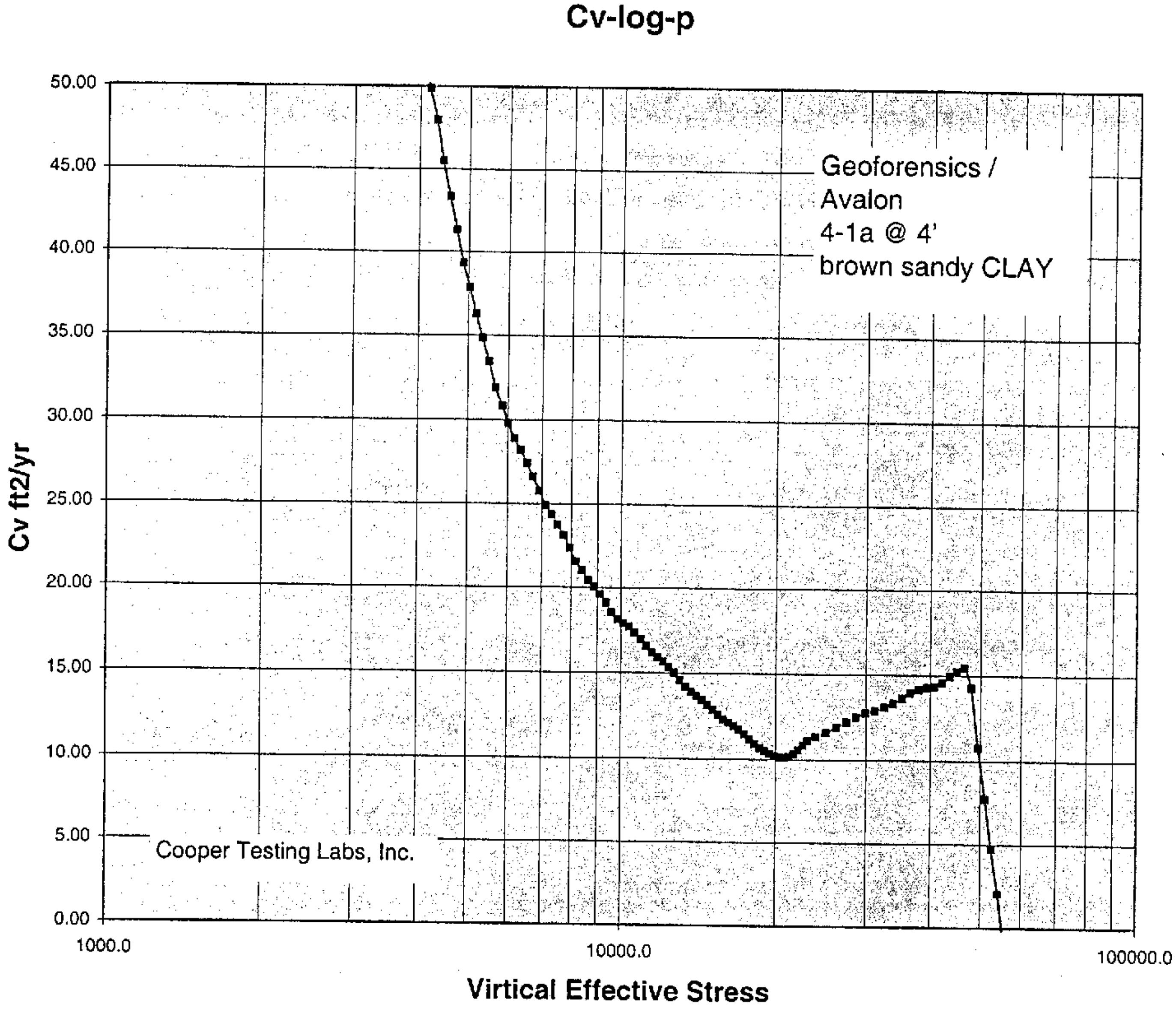
End-of-Load Summary

Pressure (ksf)	**Final Dial (in.)**	**Machine Defl. (in.)**	**C_v (ft.^{2}/day)**	**Void Ratio**	**% Compression /Swell**
start	0.00000			0.668	
0.15	0.00410	0.00000	3.11	0.661	0.4 Comprs.
0.30	0.00490	0.00000	0.10	0.660	0.5 Comprs.
0.55	0.00920	0.00000	0.13	0.653	0.9 Comprs.
1.10	0.01760	0.00000	0.06	0.639	1.8 Comprs.
2.20	0.03070	0.00000	0.06	0.617	3.1 Comprs.
4.40	0.05420	0.00000	0.05	0.578	5.4 Comprs.
8.80	0.08640	0.00000	0.04	0.524	8.6 Comprs.
17.60	0.12540	0.00000	0.03	0.459	12.5 Comprs.
35.20	0.17180	0.00000	0.03	0.382	17.2 Comprs.
70.40	0.22130	0.00000	0.01	0.299	22.1 Comprs.
17.60	0.21070	0.00000		0.317	21.1 Comprs.
4.40	0.19150	0.00000		0.349	19.2 Comprs.
1.10	0.17080	0.00000		0.383	17.1 Comprs.
0.15	0.14810	0.00000		0.421	14.8 Comprs.

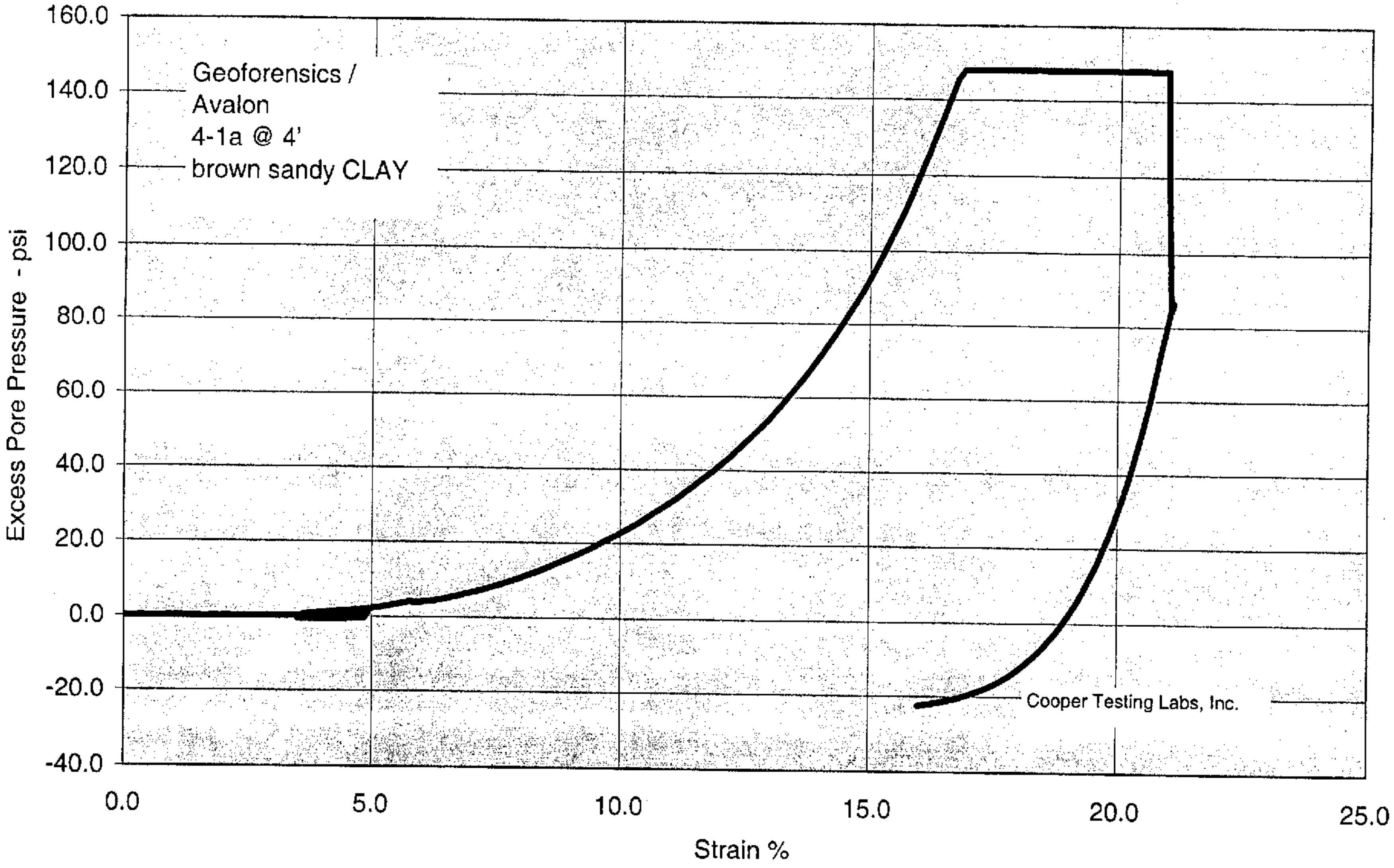
Constant Rate of Strain Consolidation Test

Cooper Testing Labs, Inc.

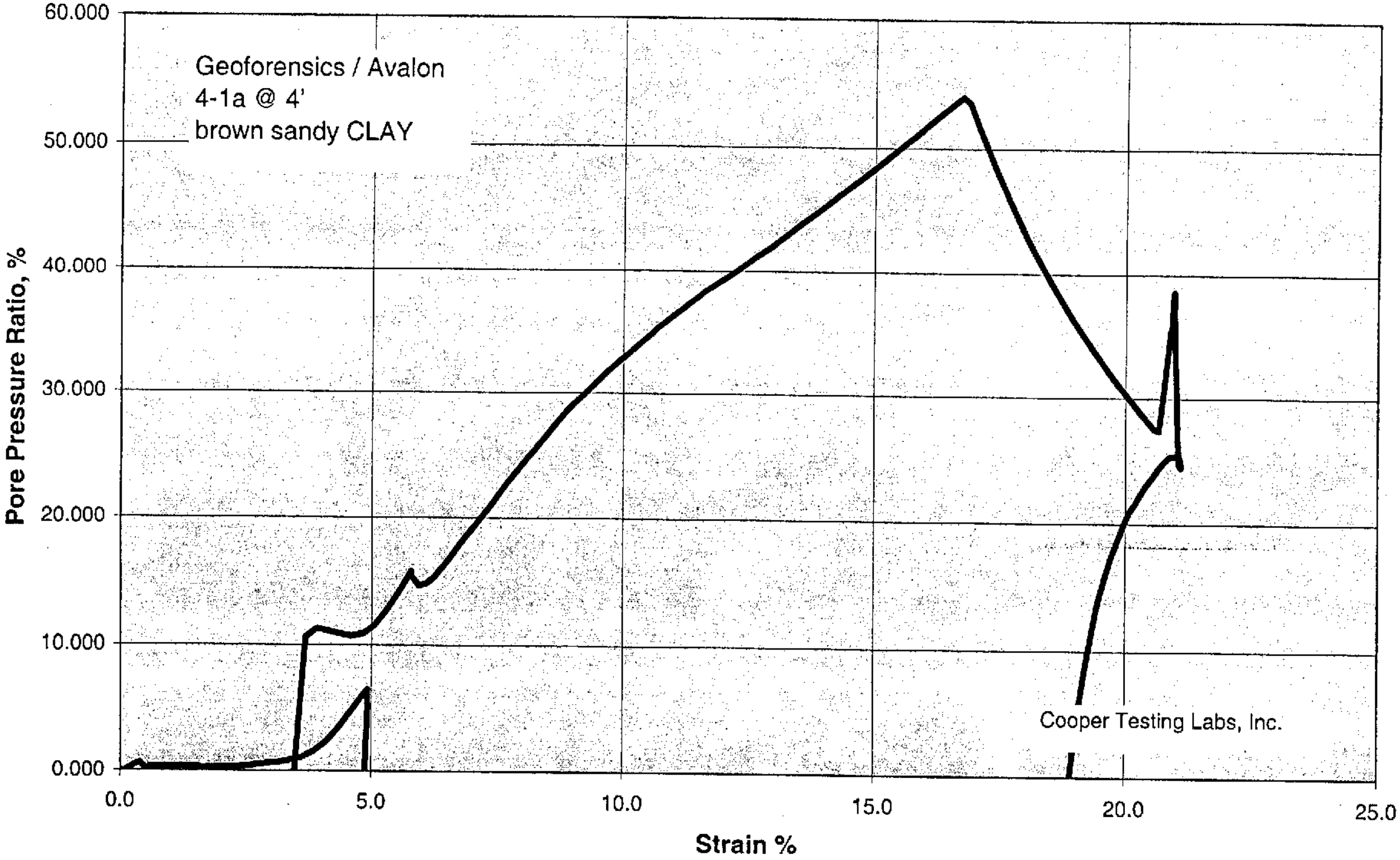


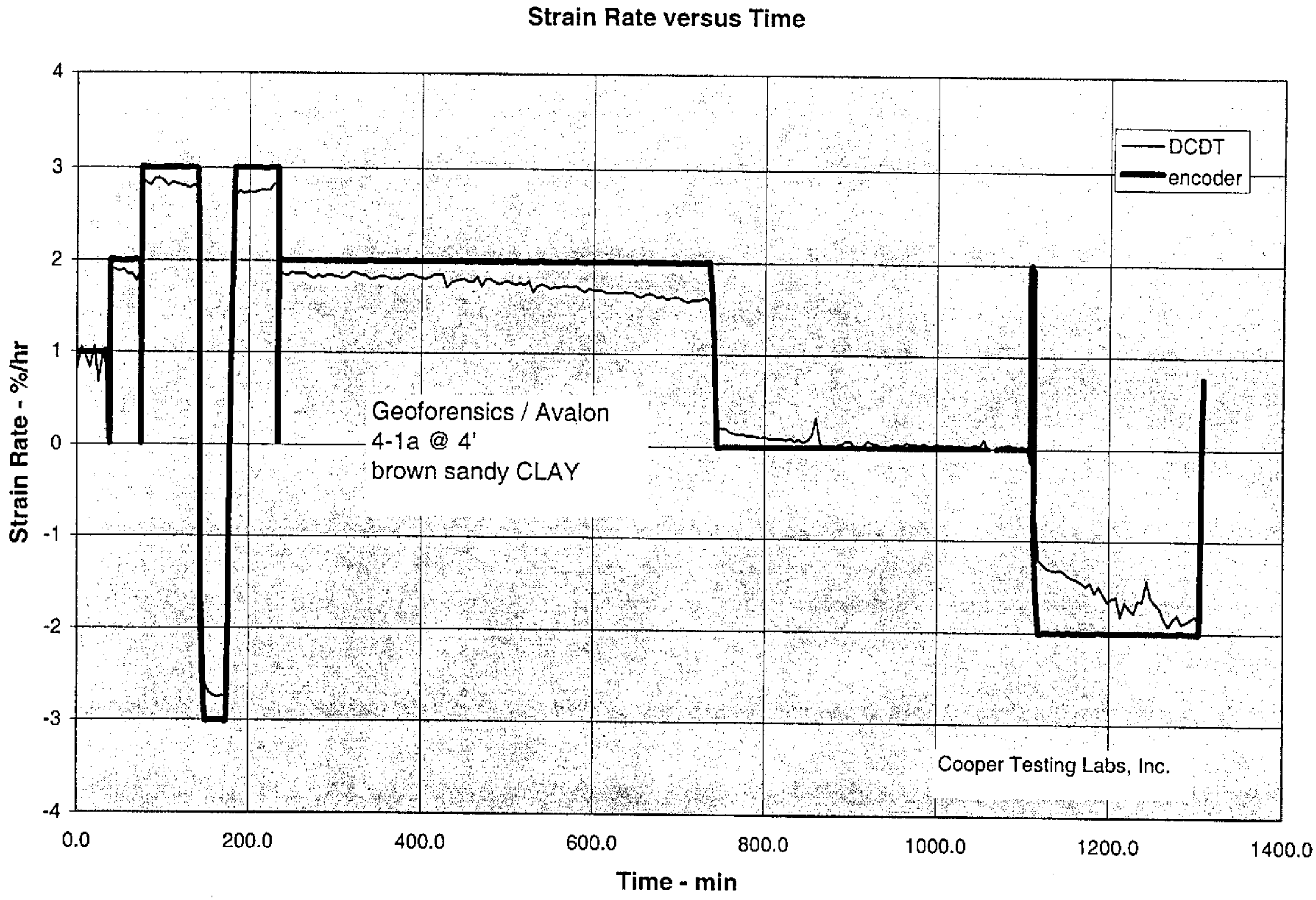

Cv-log-p
Cv ft2/yr
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
1000.0
10000.0
100000.0
Virtical Effective Stress
Geoforensics /
Avalon
4-1a @ 4'
brown sandy CLAY
Cooper Testing Labs, Inc.

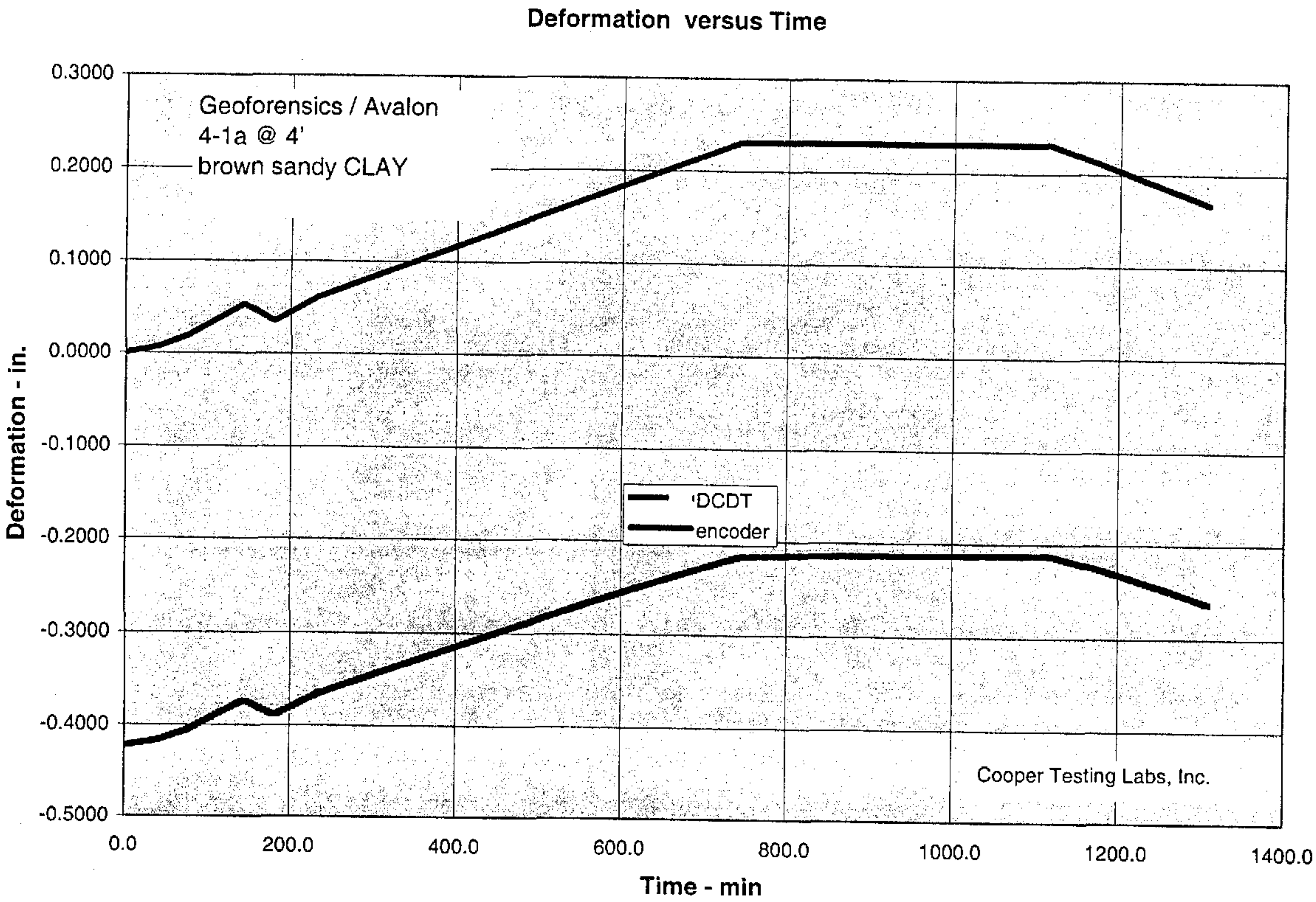
Excess Pore Pressure versus Strain

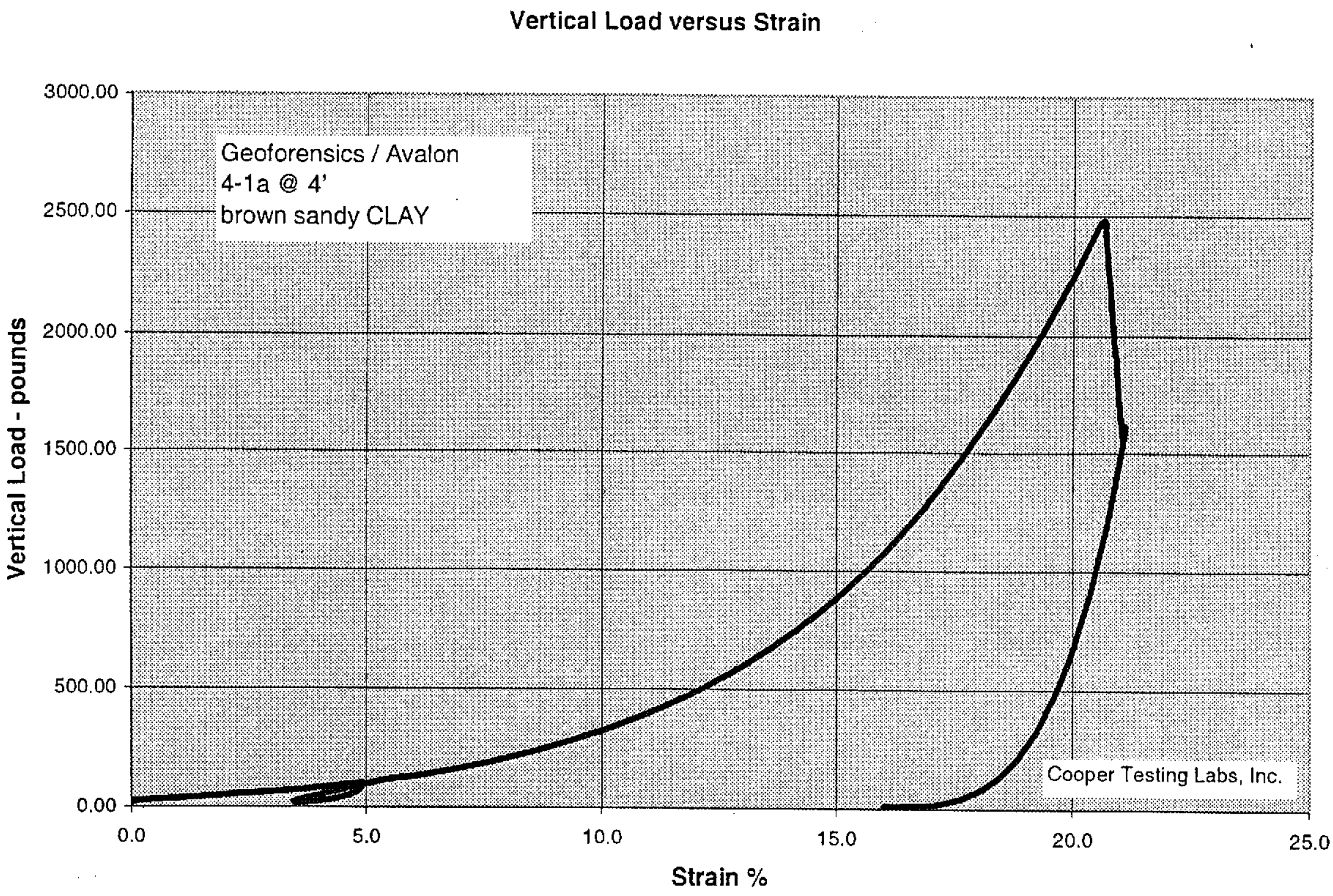


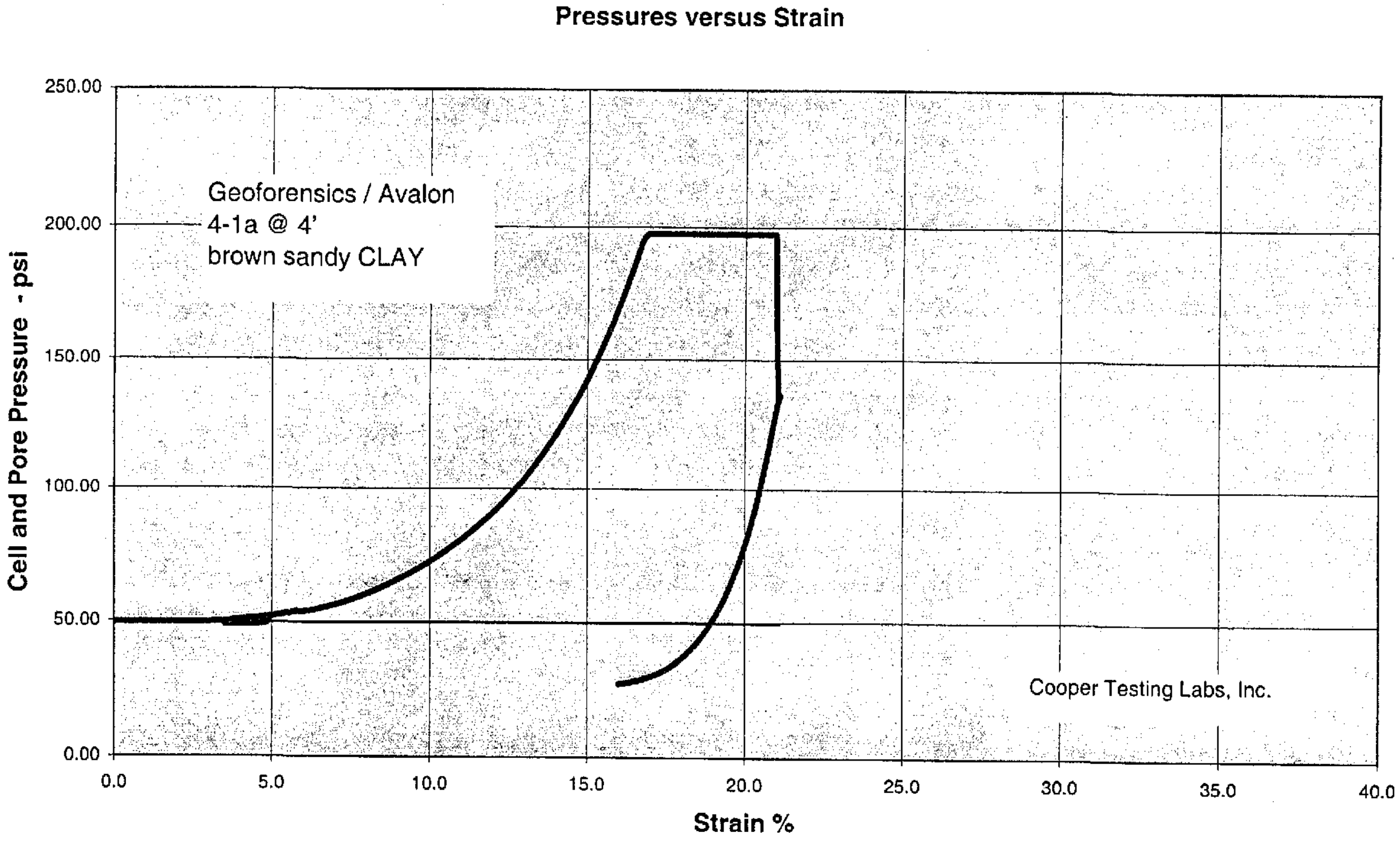
Pore Pressure Ratio




Strain Rate versus Time
Strain Rate - %/hr
4
3
2
1
0
-1
-2
-3
-4
Time - min
0.0
200.0
400.0
600.0
800.0
1000.0
1200.0
1400.0
DCDT
encoder
Geoforensics / Avalon
4-1a @ 4'
brown sandy CLAY
Cooper Testing Labs, Inc.


Deformation versus Time
Geoforensics / Avalon
4-1a @ 4'
brown sandy CLAY
'DCDT
encoder
Cooper Testing Labs, Inc.
Deformation - in.
0.3000
0.2000
0.1000
0.0000
-0.1000
-0.2000
-0.3000
-0.4000
-0.5000
Time - min
0.0
200.0
400.0
600.0
800.0
1000.0
1200.0
1400.0


Vertical Load versus Strain
Geoforensics / Avalon
4-1a @ 4'
brown sandy CLAY
Cooper Testing Labs, Inc.
Vertical Load - pounds
3000.00
2500.00
2000.00
1500.00
1000.00
500.00
0.00
0.0
5.0
10.0
15.0
20.0
25.0
Strain %

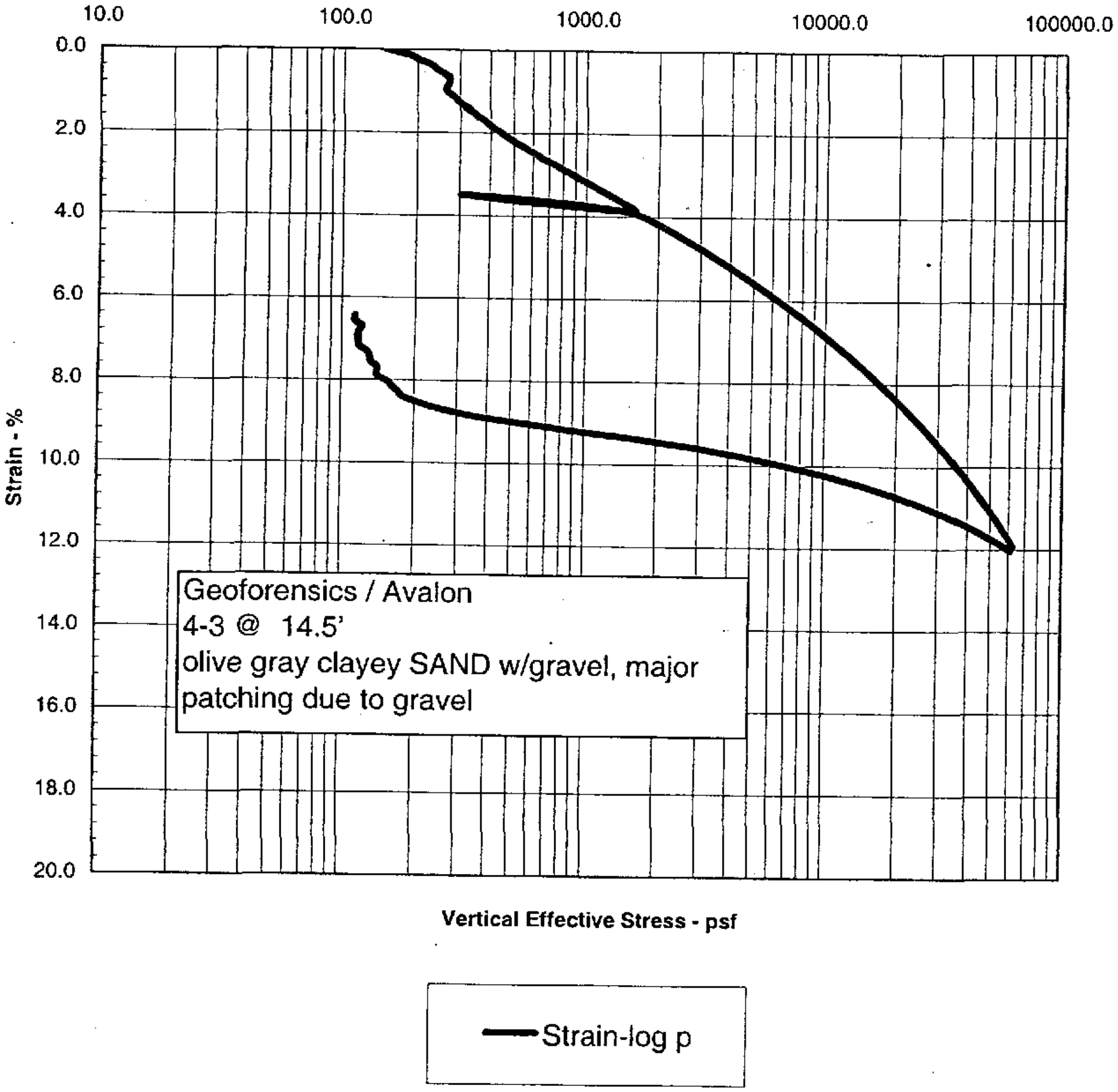
Pressures versus Strain
Geoforensics / Avalon
4-1a @ 4'
brown sandy CLAY
Cooper Testing Labs, Inc.
Cell and Pore Pressure - psi
250.00
200.00
150.00
100.00
50.00
0.00
0.0
5.0
10.0
15.0
20.0
25.0
30.0
35.0
40.0
Strain %

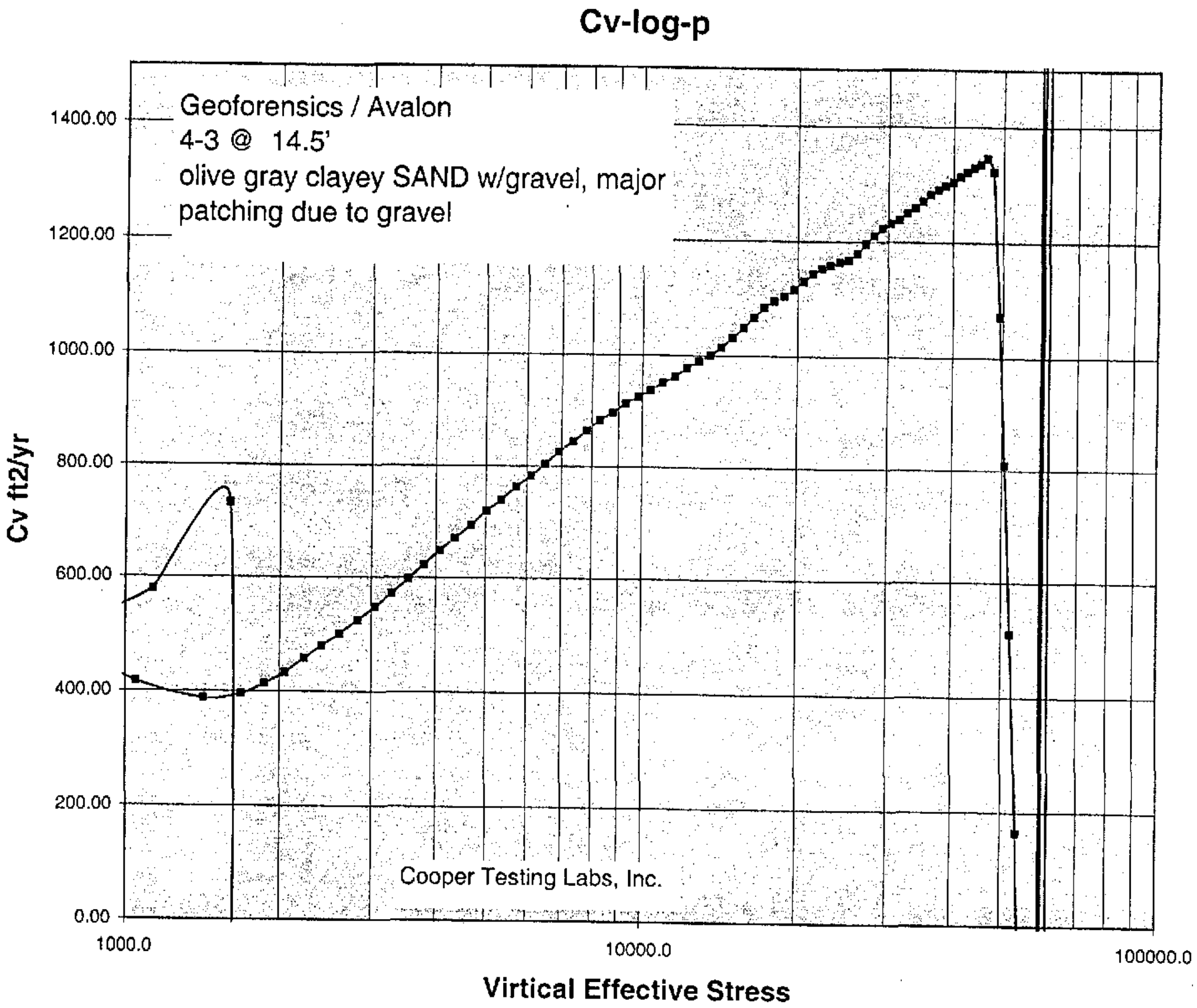
Strain-log p

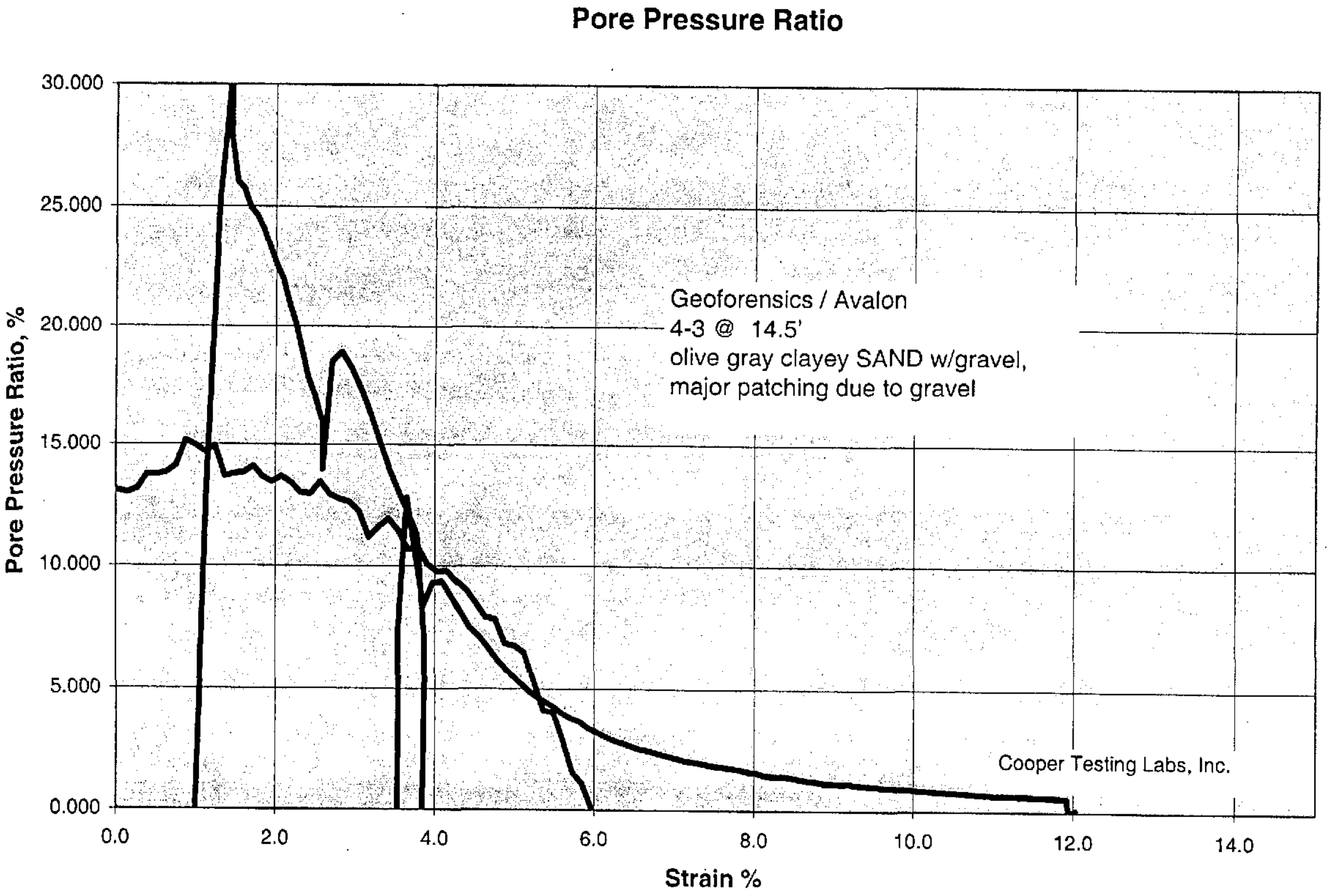
Constant Rate of Strain Consolidation Test

Cooper Testing Labs, Inc.

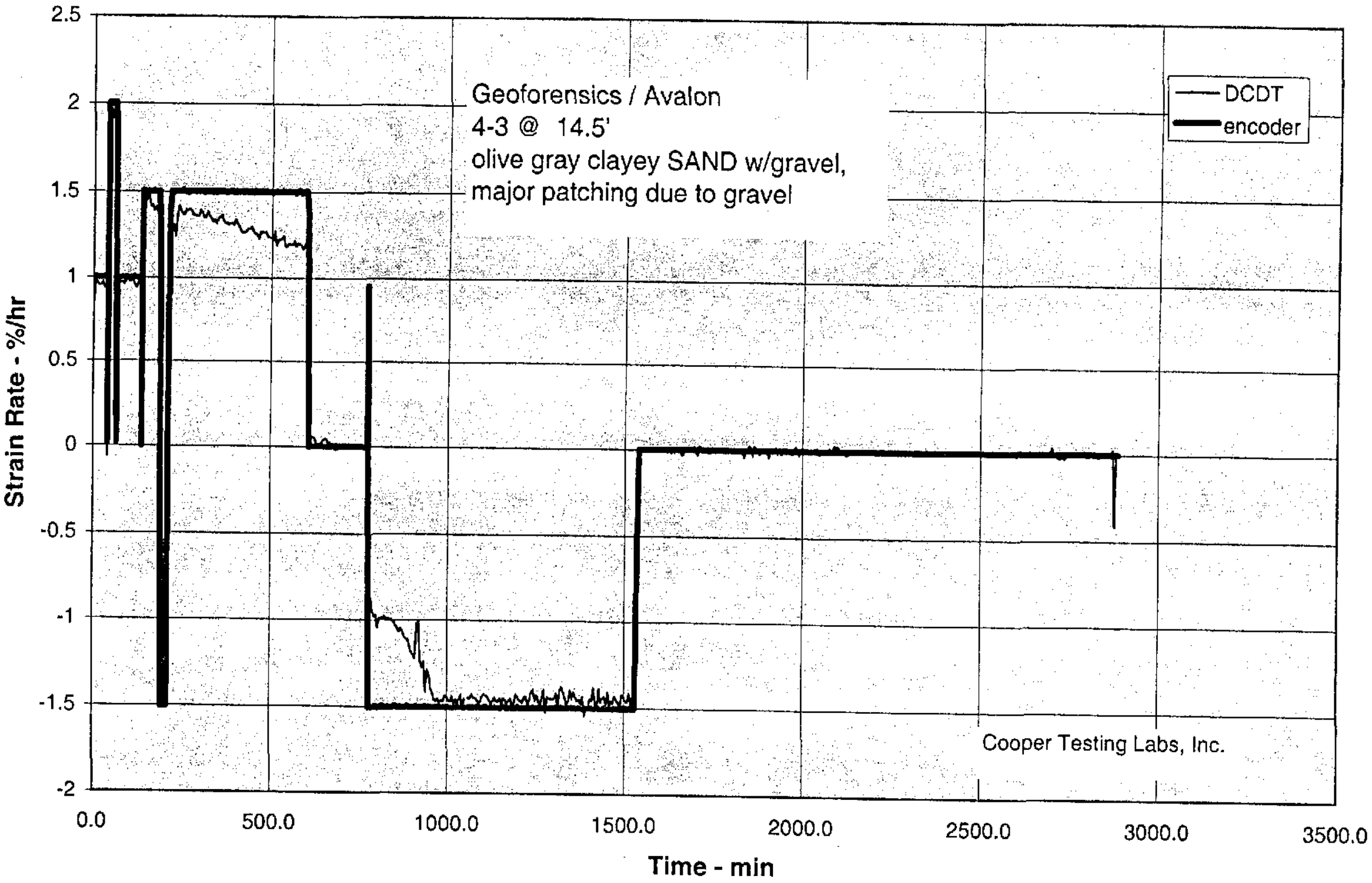
Strain-Log-P Curve

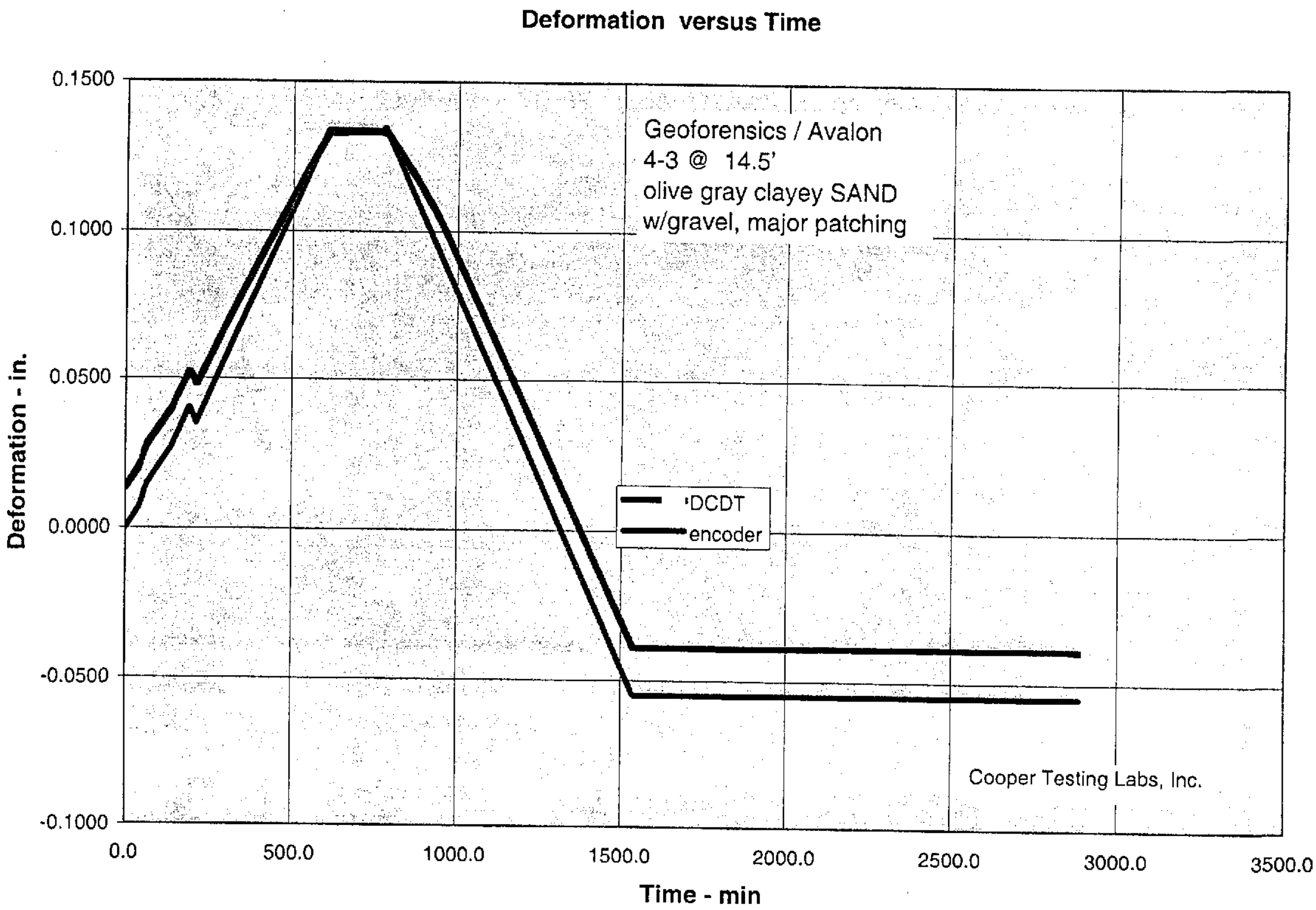



Cv-log-p
Geoforensics / Avalon
4-3 @ 14.5'
olive gray clayey SAND w/gravel, major patching due to gravel
Cv ft2/yr
1400.00
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
1000.0
10000.0
100000.0
Virtical Effective Stress
Cooper Testing Labs, Inc.


Pore Pressure Ratio
Pore Pressure Ratio, %
30.000
25.000
20.000
15.000
10.000
5.000
0.000
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
Strain %
Geoforensics / Avalon
4-3 @ 14.5'
olive gray clayey SAND w/gravel,
major patching due to gravel
Cooper Testing Labs, Inc.

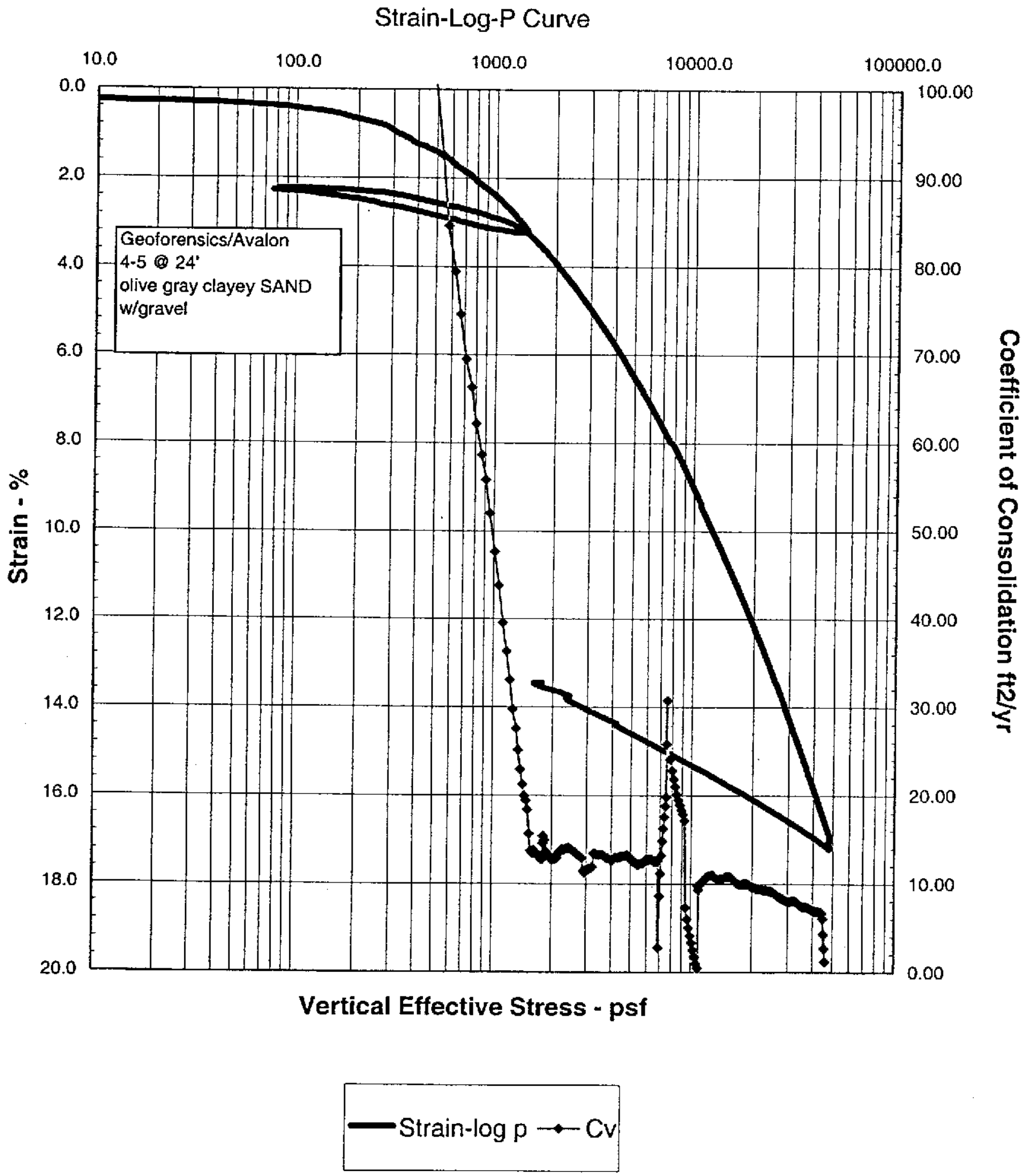
Strain Rate versus Time

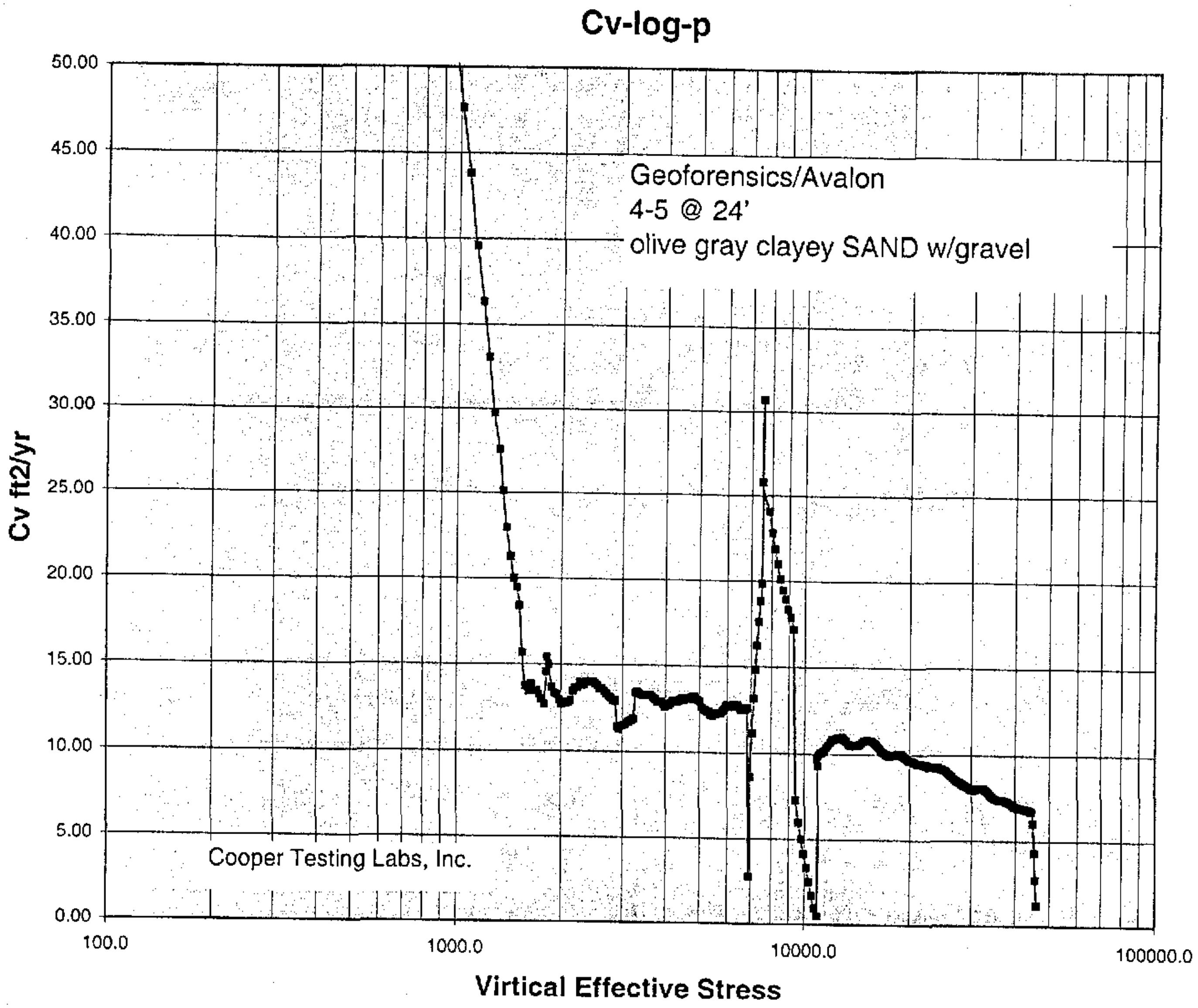



Deformation versus Time
0.1500
0.1000
0.0500
0.0000
-0.0500
-0.1000
Deformation - in.
0.0
500.0
1000.0
1500.0
2000.0
2500.0
3000.0
3500.0
Time - min
Geoforensics / Avalon
4-3 @ 14.5'
olive gray clayey SAND
w/gravel, major patching
DCDT
encoder
Cooper Testing Labs, Inc.

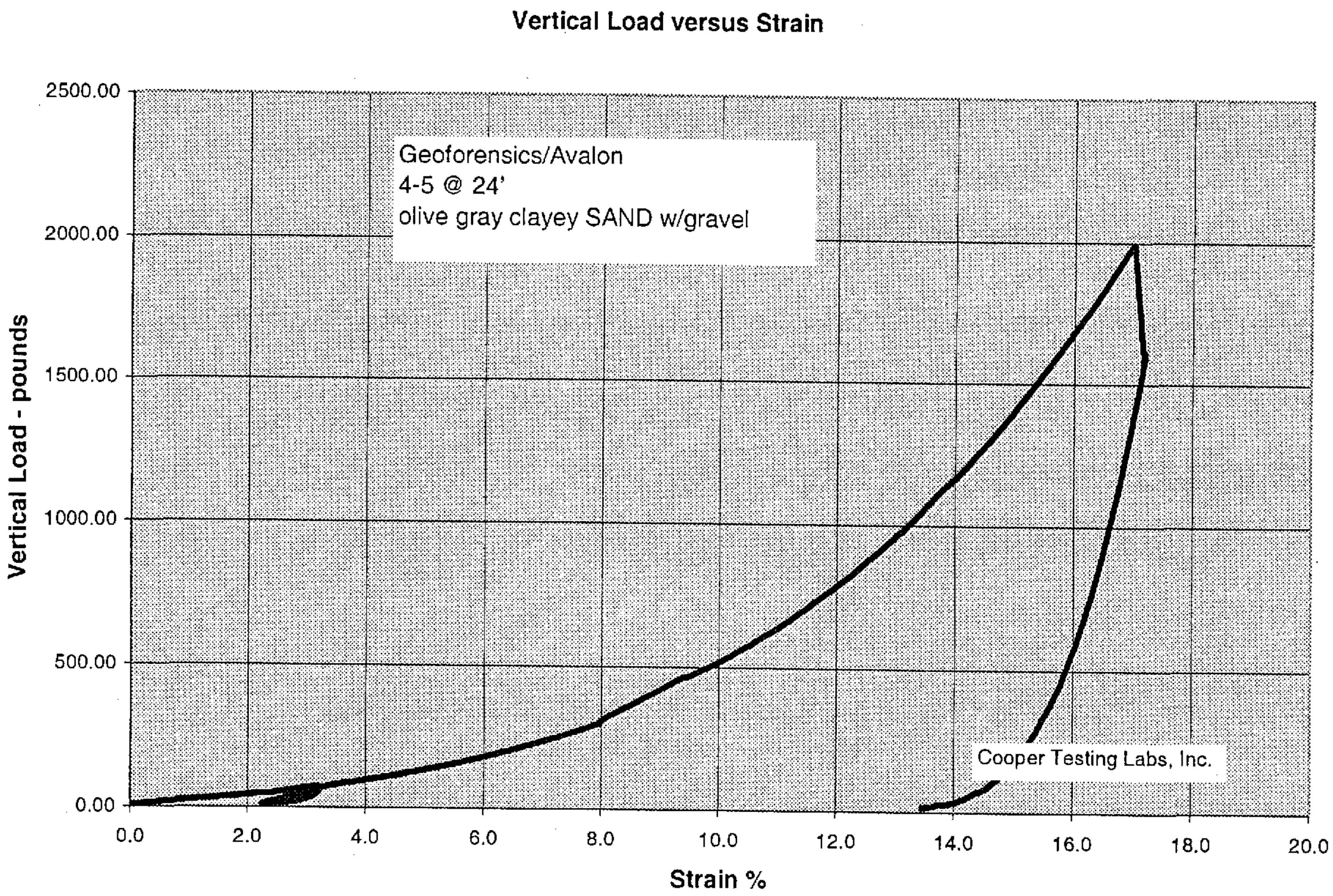
Constant Rate of Strain Consolidation Test

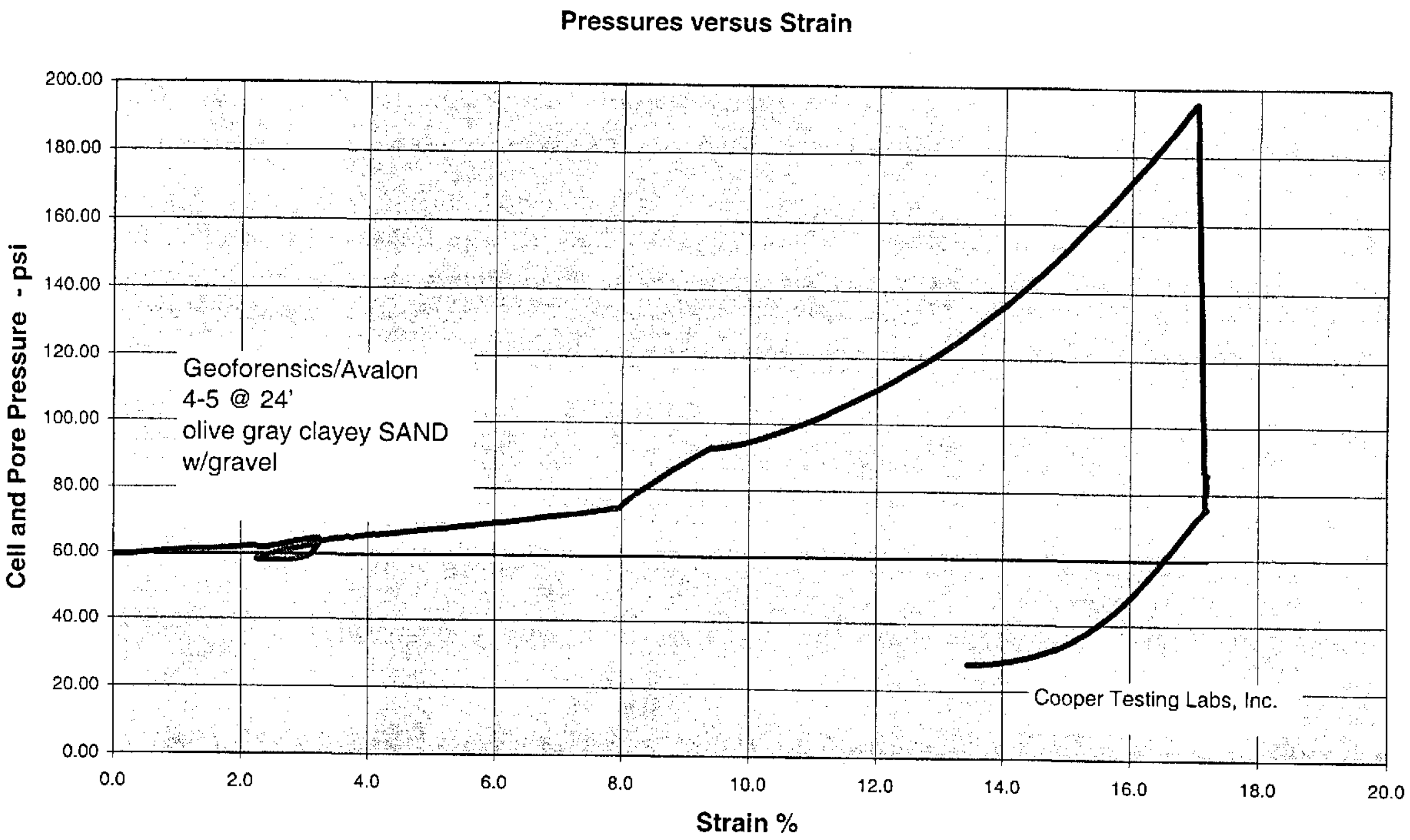
Cooper Testing Labs, Inc.

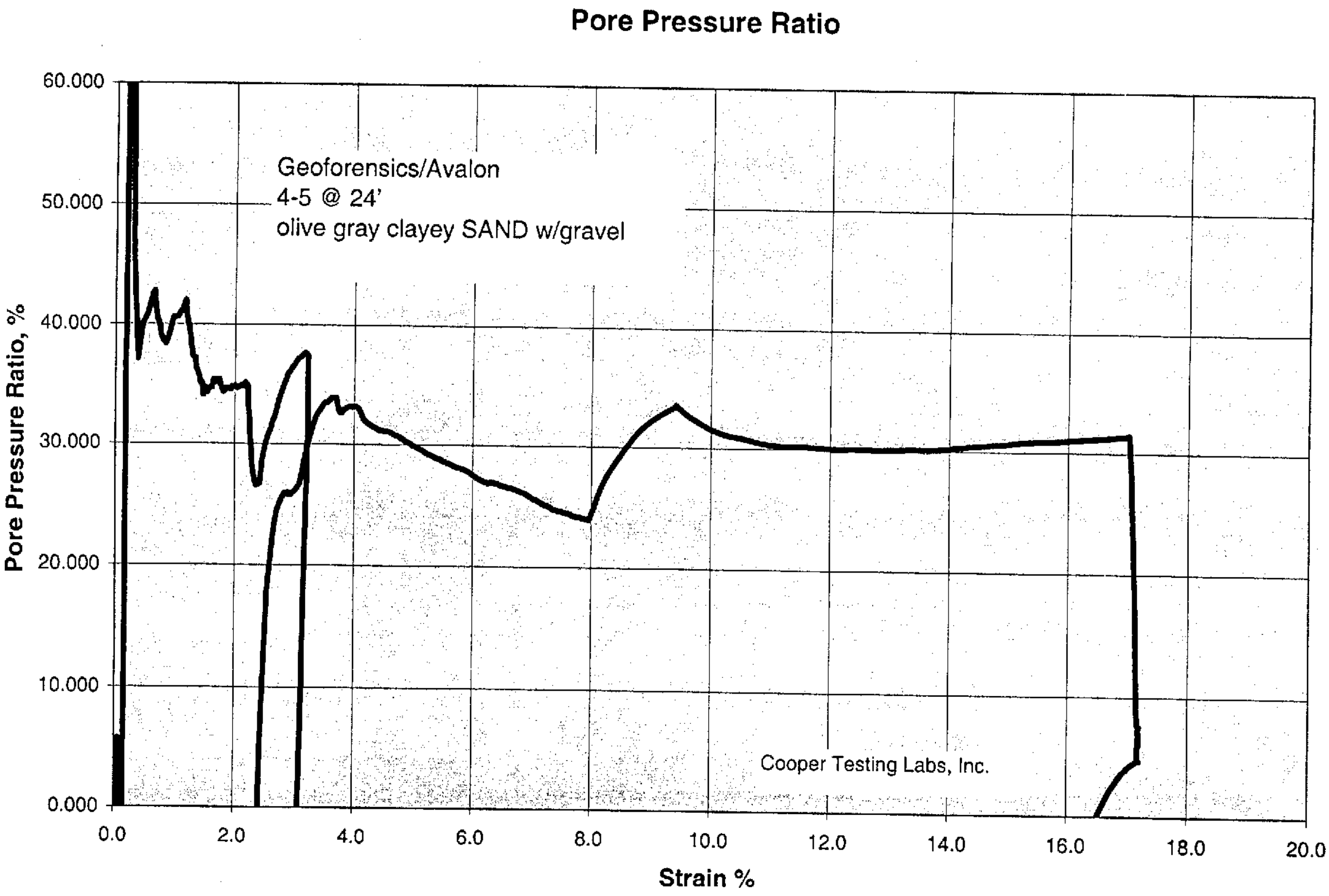




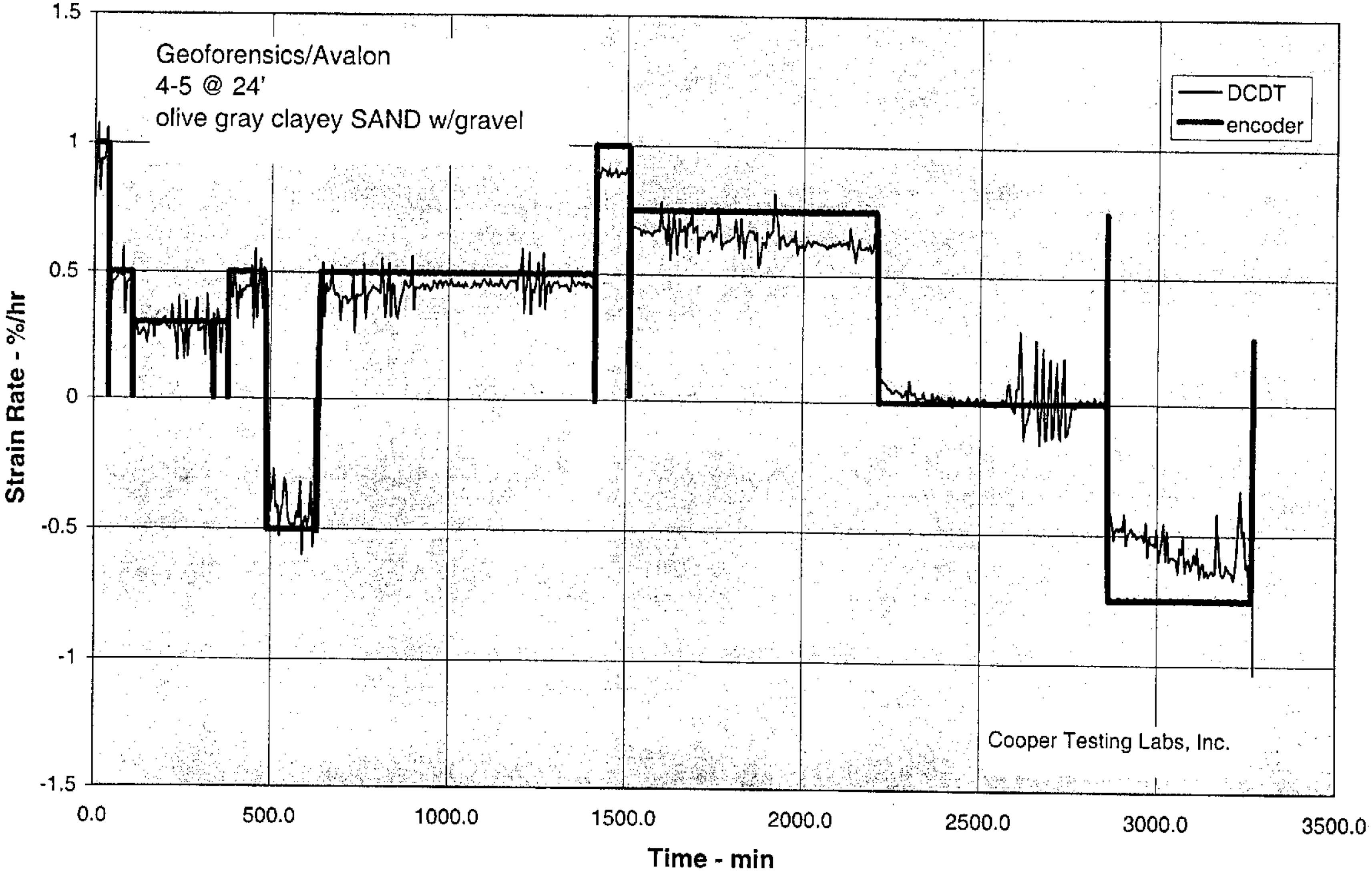
Cv-log-p
Geoforensics/Avalon
4-5 @ 24'
olive gray clayey SAND w/gravel
Cooper Testing Labs, Inc.
Cv ft2/yr
50.00
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
100.0
1000.0
10000.0
100000.0
Virtical Effective Stress

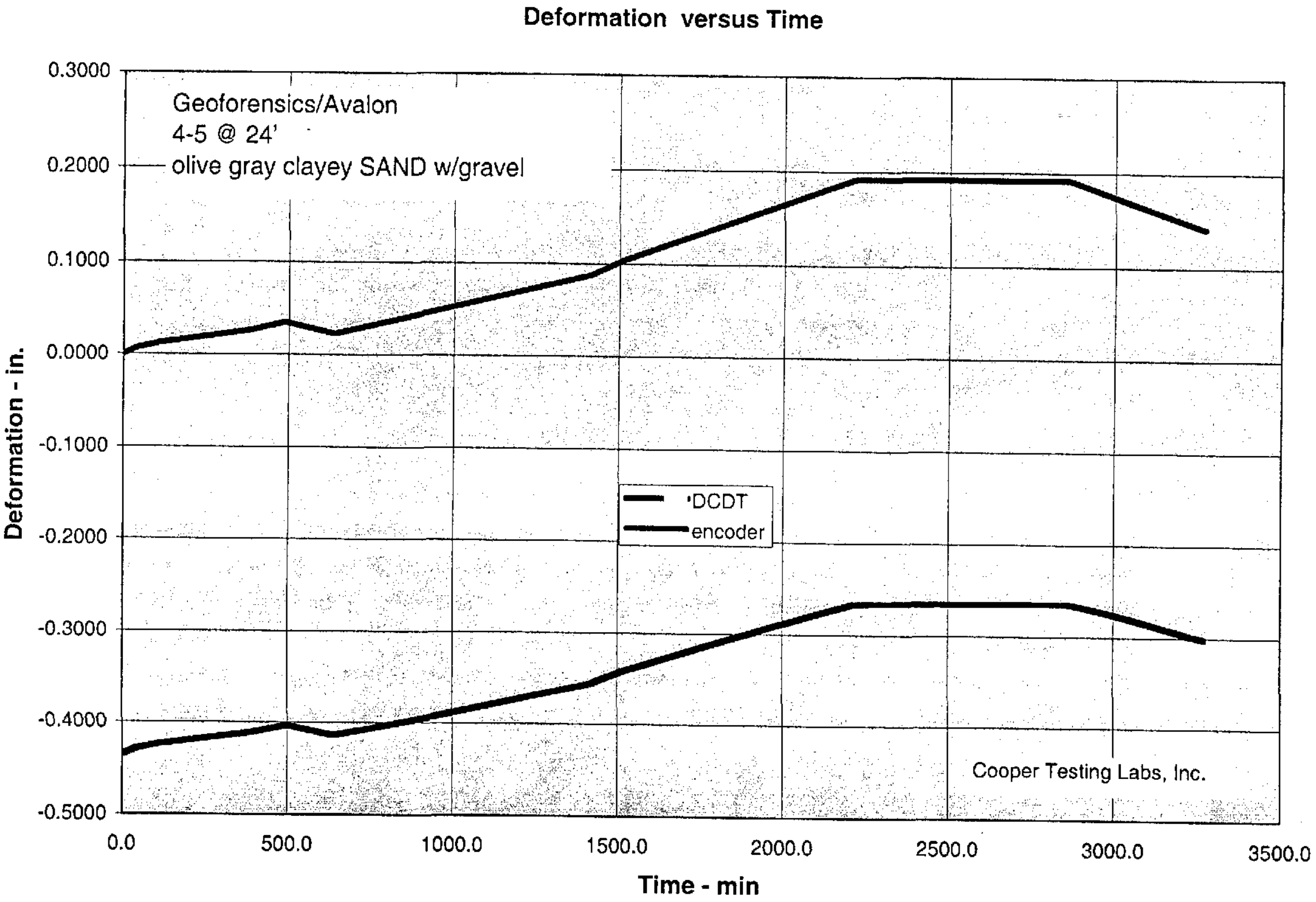

Vertical Load versus Strain
Geoforensics/Avalon
4-5 @ 24'
olive gray clayey SAND w/gravel
Cooper Testing Labs, Inc.
Vertical Load - pounds
2500.00
2000.00
1500.00
1000.00
500.00
0.00
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
Strain %

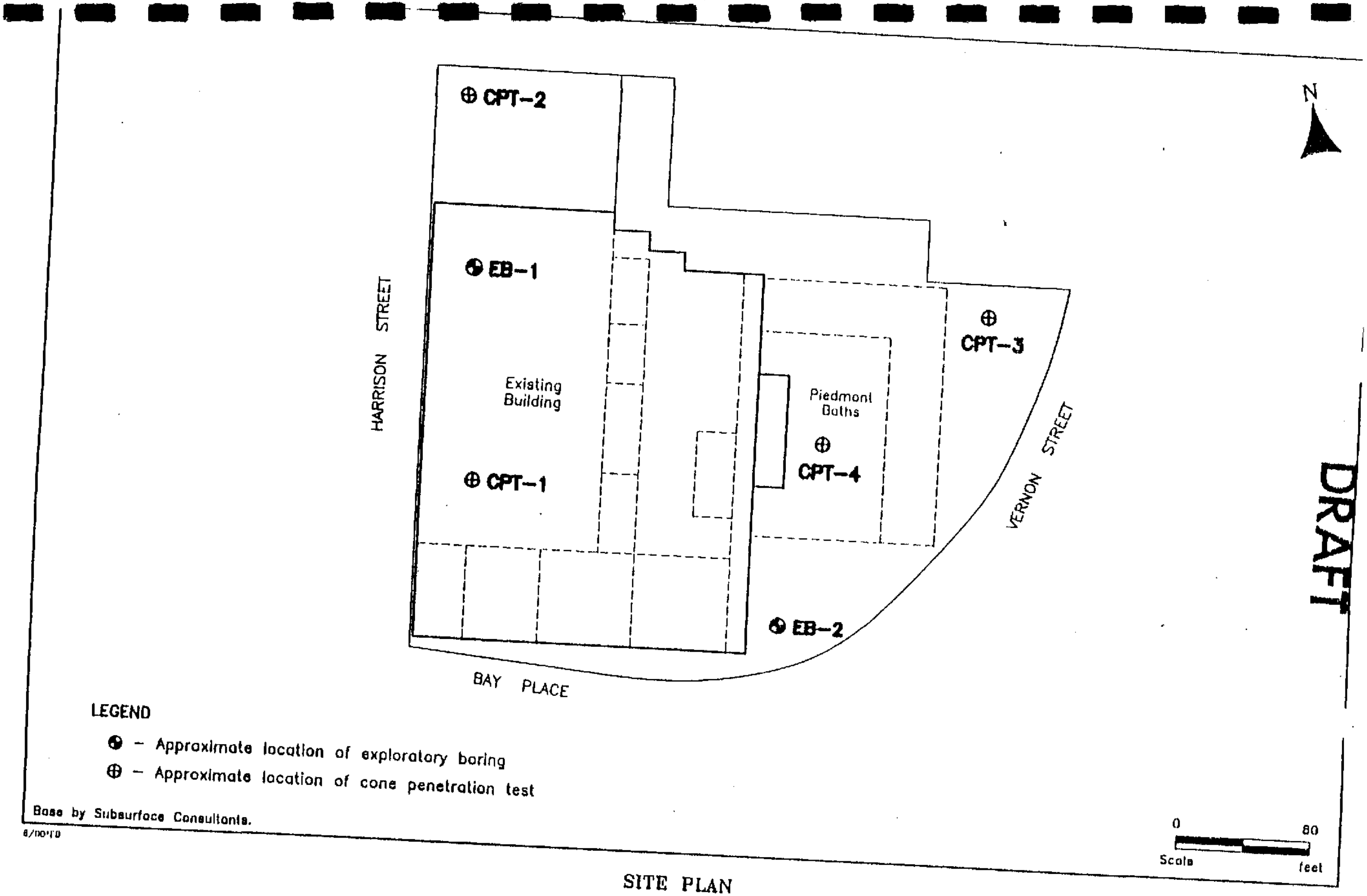

Pressures versus Strain
Cell and Pore Pressure - psi
200.00
180.00
160.00
140.00
120.00
100.00
80.00
60.00
40.00
20.00
0.00
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
Strain %
Geoforensics/Avalon
4-5 @ 24'
olive gray clayey SAND
w/gravel
Cooper Testing Labs, Inc.


Pore Pressure Ratio
Geoforensics/Avalon
4-5 @ 24'
olive gray clayey SAND w/gravel
Cooper Testing Labs, Inc.
Pore Pressure Ratio, %
60.000
50.000
40.000
30.000
20.000
10.000
0.000
0.0
2.0
4.0
6.0
8.0
10.0
12.0
14.0
16.0
18.0
20.0
Strain %

Strain Rate versus Time




Deformation versus Time
Geoforensics/Avalon
4-5 @ 24'
olive gray clayey SAND w/gravel
Deformation - in.
0.3000
0.2000
0.1000
0.0000
-0.1000
-0.2000
-0.3000
-0.4000
-0.5000
DCDT
encoder
Cooper Testing Labs, Inc.
0.0
500.0
1000.0
1500.0
2000.0
2500.0
3000.0
3500.0
Time - min



SITE PLAN
230 BAY PLACE
Oakland, California

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Environmental/Geotechnical/Engineering Services

FIGURE 2
606-71

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, rotary-wash auger drilling equipment, and truck-mounted cone penetration test (CPT) equipment. Two approximately 5-inch-diameter exploratory borings (EB-1 and EB-2) were drilled, and four CPTs (CPT-1, 2, 3 and 4) were hydraulically pushed on July 27, 2000, to maximum depths of 40 to 41 feet. The approximate locations of the exploratory borings and CPTs are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). The logs of the borings, as well as a key to the classification of the soil, are included as part of this appendix. The CPT data is also attached.

The locations of borings and CPTs were determined by approximate measurements from site and building features. Elevations of the borings were not determined. The locations of the borings and CPTs should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. Most of the soil samples were obtained with a 2.5-inch I.D. Modified-California split barrel sampler. Modified-California penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall; the sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). In addition, 2.0-inch I.D. samples were obtained using a Standard Penetration Test (SPT) split barrel sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the samplers the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

Field tests included an evaluation of the undrained shear strength of soil samples using a Torvane device, and the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

The attached boring and CPT logs and related information show subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

			TYPE		SEC ARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW		Well graded gravels, gravel-sand mixtures, little or no fines
			GP		Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVEL WITH FINES	GM		Silty gravels, gravel-sand-silt mixtures, plastic fines
			GC		Clayey gravels, gravel-sand-clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW		Well graded sands, gravelly sands, little or no fines
			SP		Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM		Silty sands, sand-silt-mixtures, non-plastic fines
			SC		Clayey sands, sand-clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50 %		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL		Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH		Inorganic clays of high plasticity, fat clays
			OH		Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT		Peat and other highly organic soils

DEFINITION OF TERMS

U.S. STANDARD SIEVE SIZE

CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4"	3"	12"
SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

☒ TERZAGHI SPLIT SPOON STANDARD PENETRATION

⧗ MODIFIED CALIFORNIA

▯ D&M UNDERWATER SAMPLER

■ SHELBY TUBE

○ NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0-1/4	0-2
SOFT	1/4-1/2	2-4
MEDIUM STIFF	1/2-1	4-8
STIFF	1-2	8-16
VERY STIFF	2-4	16-32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2-inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D-2487)

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FIGURE A-1

EXPLORATORY BORING: EB-1

Sheet 1 of 2

DRILL RIG: FAILING 1500
BORING TYPE: 4-7/8 INCH ROTARY WASH
LOGGED BY: DGJ
START DATE: 7-27-00 FINISH DATE: 7-27-00

PROJECT NO: 595-71
PROJECT: 230 BAY PLACE
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 39.0 FT.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION:

DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO 200 SIEVE
0	2½ inches concrete over 6 inches old concrete					
	SILTY CLAY (CL) [FILL] medium stiff to stiff, moist, black with brown mottles, organics, moderate plasticity	CL, FILL	6	24	100	
5	**SILTY CLAY (CL)** stiff, moist, dark gray, some sand and gravel, strong hydrocarbon odor	CL	13	20	110	
	SILTY CLAY (CL) stiff, moist, gray and brown mottled, trace gravel, moderate plasticity		25	18	114	
10	Plasticity Limit = 21, Liquid Limit = 35		37	18	110	
			23	18	111	
15						
		CL	39	17		
20	gravel and sand in cuttings					
25			14	29		
30			31	30		

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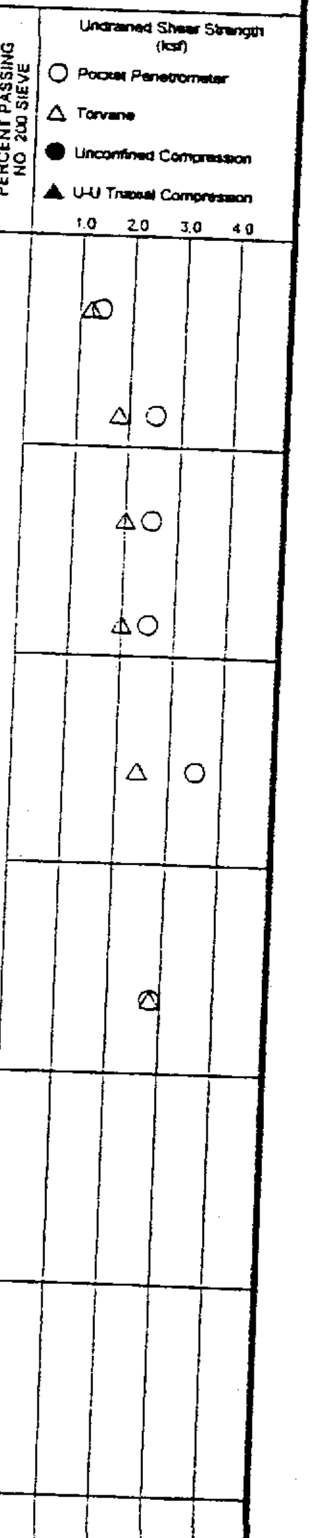

Continued Next Page

GROUND WATER OBSERVATIONS:
NO FREE GROUNDWATER ENCOUNTERED

LA CORP GDT 8/4/00 MV*FLL

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EXPLORATORY BORING: EB-1 Cont'd

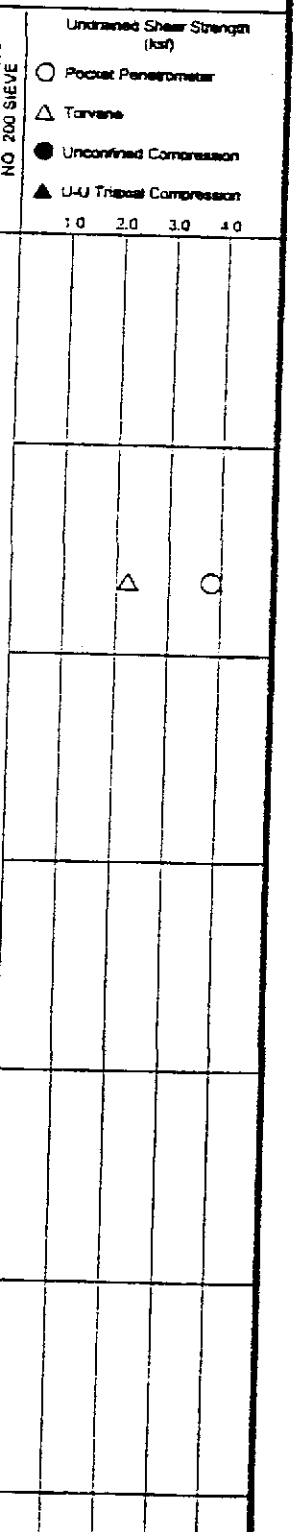
Sheet 2 of 2

DRILL RIG: FAILING 1500
BORING TYPE: 4-7/8 INCH ROTARY WASH
LOGGED BY: DGJ
START DATE: 7-27-00 FINISH DATE: 7-27-00

PROJECT NO: 595-71
PROJECT: 230 BAY PLACE
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 39.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO 200 SIEVE
	30	▨	**SILTY CLAY (CL)** stiff, moist, gray and brown mottled, trace gravel, moderate plasticity						
		▨			53	⧗	23		
	35	▨		CL					
		▨			53	⧗	25	104	
	40		Bottom of Boring at 39 feet						
	45								
	50								
	55								
	60								



DRAFT

GROUND WATER OBSERVATIONS:
NO FREE GROUNDWATER ENCOUNTERED

LA CORP GDT 8/4/00 MV' FLL

LOWNEYASSOCIATES
Environmental/Geotechnical/Engineering Services

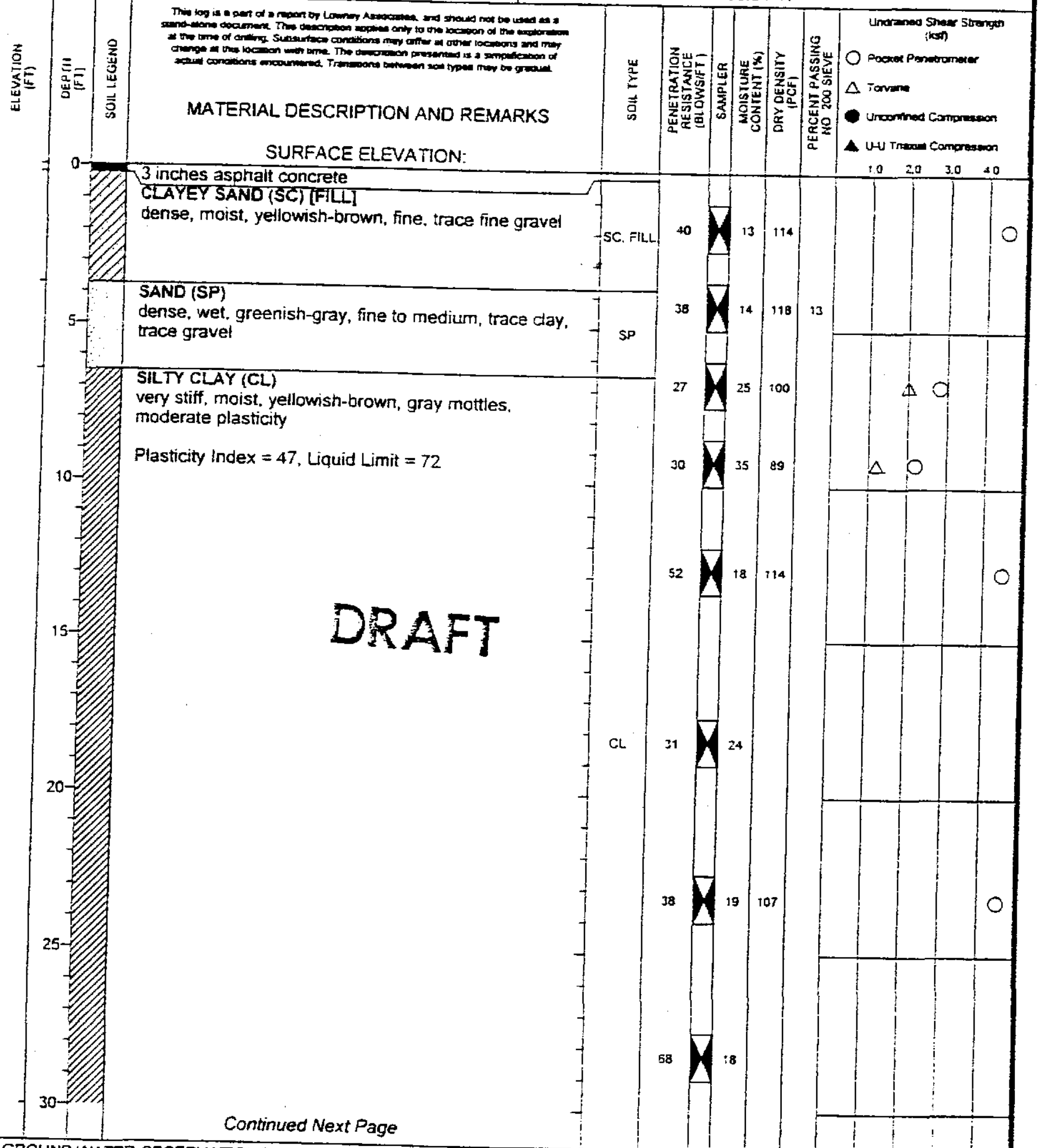
EXPLORATORY BORING: EB-2

Sheet 1 of 2

DRILL RIG: FAILING 1500
BORING TYPE: 4-7/8 INCH ROTARY WASH
LOGGED BY: DGJ
START DATE: 7-27-00 FINISH DATE: 7-27-00

PROJECT NO: 595-71
PROJECT: 230 BAY PLACE
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 39.0 FT.

This log is a part of a report by Lowney Associates, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.



MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION:

MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO 200 SIEVE
3 inches asphalt concrete					
CLAYEY SAND (SC) [FILL] dense, moist, yellowish-brown, fine, trace fine gravel	SC. FILL	40	13	114	
SAND (SP) dense, wet, greenish-gray, fine to medium, trace clay, trace gravel	SP	38	14	118	13
SILTY CLAY (CL) very stiff, moist, yellowish-brown, gray mottles, moderate plasticity Plasticity Index = 47, Liquid Limit = 72		27	25	100	
		30	35	89	
		52	18	114	
	CL	31	24		
		38	19	107	
		68	18		

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Continued Next Page

GROUND WATER OBSERVATIONS:
NO FREE GROUNDWATER ENCOUNTERED

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EXPLORATORY BORING: EB-2 Cont'd

Sheet 2 of 2

DRILL RIG: FAILING 1500
BORING TYPE: 4-7/8 INCH ROTARY WASH
LOGGED BY: DGJ
START DATE: 7-27-00 FINISH DATE: 7-27-00

PROJECT NO: 595-71
PROJECT: 230 BAY PLACE
LOCATION: OAKLAND, CA
COMPLETION DEPTH: 39.0 FT.

Undrained Shear Strength (ksf)
○ Pocket Penetrometer
△ Torvane
● Unconfined Compression
▲ U-U Triaxial Compression

ELEVATION (FT)	DEPTH (FT)	SOIL LEGEND	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO 200 SIEVE	Undrained Shear Strength (ksf) 1.0 2.0 3.0 4.0
	30	▨	**SILTY CLAY (CL)** very stiff, moist, yellowish-brown, gray mottles, moderate plasticity							
		▨			51	☒	25	100		○
	35	▨		CL						
		▨			84	☒	20			
	40		Bottom of Boring at 39 feet							
	45									
	50									
	55									
	60									

DRAFT

LA CORP GDT 8/4/00 MV' F11

GROUND WATER OBSERVATIONS:
NO FREE GROUNDWATER ENCOUNTERED

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Environmental/Geotechnical/Engineering Services

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214(LNY-73)

CPT NO.: CPT-1
DATE : 07-27-2000
Groundwater measured at 5.9 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0.50	115.21	3.035	2.6	38	61	0.06	42	----	Silty SAND to Sandy SILT	130-140
1.00	14.91	0.853	5.7	15	24	0.12	----	1.98	CLAY	120-130
1.50	26.49	0.542	2.0	11	17	0.19	----	3.52	Sandy SILT to Clayey SILT	''
2.00	13.36	0.784	5.9	13	21	0.25	----	1.76	CLAY	''
2.50	7.38	0.472	6.4	7	12	0.31	----	1.45	''	110-120
3.00	5.85	0.379	6.5	6	9	0.36	----	1.13	''	100-110
3.50	4.99	0.501	10.0	5	8	0.42	----	0.96	Organic Material	110-120
4.00	5.44	0.412	7.6	5	9	0.48	----	1.04	CLAY	100-110
4.50	5.18	0.400	7.7	5	8	0.53	----	0.98	''	''
5.00	7.45	0.439	5.9	7	12	0.59	----	1.43	''	110-120
5.50	2.29	0.170	7.4	2	4	0.64	----	0.39	Organic Material	90-100
6.00	7.81	0.498	6.4	8	13	0.69	----	1.49	CLAY	110-120
6.50	7.51	0.481	6.4	8	12	0.75	----	1.43	''	''
7.00	4.34	0.290	6.7	4	7	0.80	----	0.79	''	100-110
7.50	5.05	0.283	5.6	5	8	0.85	----	0.92	''	''
8.00	3.64	0.216	6.0	4	6	0.90	----	0.64	''	90-100
8.50	3.15	0.201	6.4	3	5	0.95	----	0.53	''	''
9.00	9.78	0.134	1.4	5	8	1.00	----	1.55	Clayey SILT to Silty CLAY	''
9.50	3.27	0.104	3.2	3	5	1.05	----	0.55	CLAY	''
10.00	6.40	0.240	3.7	6	10	1.09	----	1.17	''	100-110
10.50	12.09	0.123	1.0	5	7	1.15	----	1.54	Sandy SILT to Clayey SILT	90-100
11.00	5.36	0.073	1.4	3	4	1.19	----	0.95	Sensitive Fine Grained	''
11.50	3.80	0.121	3.2	4	6	1.24	----	0.64	CLAY	''
12.00	3.23	0.120	3.7	3	5	1.29	----	0.52	''	''
12.50	3.76	0.141	3.7	4	6	1.34	----	0.62	''	''
13.00	7.16	0.185	2.6	5	7	1.39	----	1.29	Silty CLAY to CLAY	100-110
13.50	3.11	0.091	2.9	3	5	1.44	----	0.48	CLAY	90-100
14.00	3.05	0.087	2.9	3	4	1.48	----	0.46	''	85-90
14.50	5.22	0.137	2.6	5	7	1.53	----	0.89	''	90-100
15.00	6.89	0.121	1.8	3	5	1.57	----	1.22	Clayey SILT to Silty CLAY	''
15.50	3.62	0.088	2.4	4	5	1.62	----	0.56	CLAY	''
16.00	9.88	0.201	2.0	5	7	1.67	----	1.51	Clayey SILT to Silty CLAY	100-110
16.50	13.30	0.362	2.7	7	9	1.73	----	1.66	''	110-120
17.00	13.60	0.359	2.6	7	9	1.79	----	1.69	''	''
17.50	15.48	0.451	2.9	8	10	1.85	----	1.94	''	120-130
18.00	16.96	0.541	3.2	8	11	1.91	----	2.13	''	''
18.50	17.36	0.546	3.1	9	11	1.97	----	2.18	''	''
19.00	16.69	0.494	3.0	8	11	2.04	----	2.09	''	''
19.50	16.89	0.501	3.0	8	11	2.10	----	2.11	''	''
20.00	13.72	1.163	8.5	14	17	2.16	----	1.69	CLAY	130-140
20.50	14.09	0.603	4.3	14	18	2.23	----	1.73	''	120-130
21.00	14.29	0.420	2.9	7	9	2.29	----	1.75	Clayey SILT to Silty CLAY	''
21.50	15.14	0.471	3.1	8	9	2.35	----	1.86	''	''
22.00	13.50	0.413	3.1	9	11	2.41	----	1.64	Silty CLAY to CLAY	''
22.50	14.11	0.417	3.0	7	8	2.48	----	1.72	Clayey SILT to Silty CLAY	''
23.00	11.99	0.385	3.2	8	9	2.53	----	1.79	Silty CLAY to CLAY	110-120
23.50	15.99	0.527	3.3	11	12	2.60	----	1.96	''	120-130
24.00	21.89	1.414	6.5	22	25	2.66	----	2.74	CLAY	130-140
24.50	81.73	2.408	2.9	33	37	2.73	----	10.72	Sandy SILT to Clayey SILT	''
25.00	136.92	2.588	1.9	46	51	2.80	40	----	Silty SAND to Sandy SILT	''
25.50	63.34	2.455	3.9	32	35	2.86	----	8.25	Clayey SILT to Silty CLAY	''
26.00	20.11	1.043	5.2	20	22	2.93	----	2.49	CLAY	''
26.50	13.30	0.839	6.3	13	14	3.00	----	1.57	''	120-130
27.00	12.91	0.475	3.7	9	9	3.06	----	1.52	Silty CLAY to CLAY	''
27.50	14.07	0.530	3.8	9	10	3.12	----	1.67	''	''
28.00	22.49	0.829	3.7	15	16	3.18	----	2.79	''	130-140
28.50	26.00	1.230	4.7	26	27	3.25	----	3.25	CLAY	''
29.00	32.62	1.254	3.8	16	17	3.32	----	4.13	Clayey SILT to Silty CLAY	''
29.50	36.79	1.326	3.6	18	19	3.39	----	4.68	''	''
30.00	37.22	1.535	4.1	25	25	3.45	----	4.73	Silty CLAY to CLAY	''
30.50	27.22	1.200	4.4	18	18	3.52	----	3.39	''	''
31.00	18.18	1.011	5.6	18	18	3.59	----	2.19	CLAY	''
31.50	16.53	0.889	5.4	17	17	3.65	----	1.96	''	120-130
32.00	23.54	0.881	3.7	16	16	3.72	----	2.89	Silty CLAY to CLAY	130-140
32.50	26.00	0.942	3.6	13	13	3.79	----	3.21	Clayey SILT to Silty CLAY	''

John Sarmiento & Associates
Cone Penetration Testing Service

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214(LNY-73)

CPT NO.: CPT-1
DATE : 07-27-2000
Groundwater measured at 5.9 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
33.00	28.94	0.956	3.3	14	14	3.85	----	3.60	,,	,,
33.50	29.92	0.805	2.7	12	12	3.92	----	3.73	Sandy SILT to Clayey SILT	,,
34.00	30.66	0.922	3.0	15	15	3.99	----	3.82	Clayey SILT to Silty CLAY	,,
34.50	33.60	1.274	3.8	17	17	4.06	----	4.21	,,	,,
35.00	40.47	1.739	4.3	27	27	4.12	----	5.12	Silty CLAY to CLAY	,,
35.50	36.79	1.372	3.7	18	18	4.19	----	4.63	Clayey SILT to Silty CLAY	,,
36.00	33.85	1.265	3.7	17	17	4.26	----	4.23	,,	,,
36.50	36.97	1.253	3.4	18	18	4.33	----	4.64	,,	,,
37.00	37.77	1.144	3.0	19	19	4.39	----	4.74	,,	,,
37.50	39.00	1.090	2.8	16	15	4.46	----	4.90	Sandy SILT to Clayey SILT	,,
38.00	39.49	1.120	2.8	16	16	4.53	----	4.96	,,	,,
38.50	43.17	1.126	2.6	17	17	4.60	----	5.45	,,	,,
39.00	39.67	1.046	2.6	16	15	4.66	----	4.98	,,	,,
39.50	36.54	1.018	2.8	15	14	4.73	----	4.56	,,	,,
40.00	64.01	2.454	3.8	32	30	4.80	----	8.22	Clayey SILT to Silty CLAY	,,
40.50	78.18	3.521	4.5	78	73	4.87	----	10.10	Very Stiff Fine Grained *	,,
41.00	81.91	4.816	5.9	82	75	4.93	----	10.59	,,	,,

DEPTH = Sampling interval (2 inches)
Qc = Tip bearing resistance
Fs = Sleeve friction resistance
Rf = Tip/Sleeve ratio
SPT = Equivalent Standard Penetration Test*
TotStr = Total Stress using est. density**
Phi = Soil friction angle***
Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf) (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)

References: * Robertson and Campanella, 1988
** Olsen, 1989 ***Durgunoglu & Mitchell, 1975

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214(LNY-73)

CPT NO.: CPT-2
DATE : 07-27-2000
Groundwater measured at 0.1 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0.50	4.83	0.214	4.4	5	8	0.06	----	0.96	CLAY	100-110
1.00	23.40	0.417	1.8	9	15	0.12	----	3.11	Sandy SILT to Clayey SILT	120-130
1.50	82.15	3.537	4.3	41	66	0.18	----	10.94	Clayey SILT to Silty CLAY	130-140
2.00	9.72	0.557	5.7	10	16	0.25	----	1.60	CLAY	120-130
2.50	8.20	0.432	5.3	8	13	0.31	----	1.61	,,	110-120
3.00	13.05	0.553	4.2	13	21	0.37	----	1.72	,,	120-130
3.50	15.08	0.558	3.7	10	16	0.43	----	1.98	Silty CLAY to CLAY	,,
4.00	14.71	1.133	7.7	15	24	0.49	----	1.93	CLAY	130-140
4.50	13.26	1.032	7.8	13	21	0.56	----	1.73	,,	120-130
5.00	16.53	0.931	5.6	17	26	0.62	----	2.16	,,	,,
5.50	22.41	1.288	5.7	22	36	0.69	----	2.94	,,	130-140
6.00	24.19	1.202	5.0	24	39	0.75	----	3.17	,,	,,
6.50	23.15	1.214	5.2	23	37	0.82	----	3.03	,,	,,
7.00	23.52	1.284	5.5	24	38	0.89	----	3.08	,,	,,
7.50	21.01	1.110	5.3	21	34	0.96	----	2.74	,,	,,
8.00	16.88	1.021	6.0	17	27	1.02	----	2.18	,,	,,
8.50	18.37	0.928	5.1	18	29	1.09	----	2.38	,,	,,
9.00	37.07	1.975	5.3	37	59	1.16	----	4.87	,,	,,
9.50	17.89	0.915	5.1	18	29	1.22	----	2.30	,,	120-130
10.00	36.27	2.402	6.6	36	58	1.29	----	4.75	,,	130-140
10.50	15.20	1.094	7.2	15	24	1.36	----	1.94	,,	,,
11.00	20.57	0.861	4.2	14	22	1.42	----	2.65	Silty CLAY to CLAY	,,
11.50	23.88	1.309	5.5	24	38	1.49	----	3.08	CLAY	,,
12.00	18.31	1.059	5.8	18	29	1.56	----	2.34	,,	,,
12.50	29.52	1.561	5.3	30	45	1.62	----	3.83	,,	,,
13.00	43.26	2.007	4.6	29	43	1.69	----	5.66	Silty CLAY to CLAY	,,
13.50	27.75	1.207	4.3	18	27	1.76	----	3.58	,,	,,
14.00	27.44	1.173	4.3	18	26	1.83	----	3.54	,,	,,
14.50	26.46	1.004	3.8	18	25	1.89	----	3.40	,,	,,
15.00	27.19	1.000	3.7	14	19	1.96	----	3.50	Clayey SILT to Silty CLAY	,,
15.50	29.89	1.057	3.5	15	20	2.03	----	3.85	,,	,,
16.00	30.94	1.069	3.5	15	21	2.10	----	3.99	,,	,,
16.50	28.18	0.988	3.5	14	19	2.16	----	3.61	,,	,,
17.00	24.56	1.314	5.4	25	32	2.23	----	3.13	CLAY	,,
17.50	28.05	1.154	4.1	19	24	2.30	----	3.59	Silty CLAY to CLAY	,,
18.00	30.14	1.096	3.6	15	19	2.37	----	3.86	Clayey SILT to Silty CLAY	,,
18.50	27.56	1.059	3.8	18	23	2.43	----	3.51	Silty CLAY to CLAY	,,
19.00	35.35	1.267	3.6	18	22	2.50	----	4.55	Clayey SILT to Silty CLAY	,,
19.50	35.78	1.426	4.0	18	22	2.57	----	4.60	,,	,,
20.00	38.05	1.434	3.8	19	23	2.64	----	4.90	,,	,,
20.50	37.06	1.519	4.1	25	29	2.70	----	4.76	Silty CLAY to CLAY	,,
21.00	31.36	1.367	4.4	21	24	2.77	----	4.00	,,	,,
21.50	30.63	1.256	4.1	20	23	2.84	----	3.89	,,	,,
22.00	28.17	1.063	3.8	14	16	2.91	----	3.56	Clayey SILT to Silty CLAY	,,
22.50	28.97	1.034	3.6	14	16	2.97	----	3.66	,,	,,
23.00	28.11	1.010	3.6	14	16	3.04	----	3.55	,,	,,
23.50	26.03	1.119	4.3	17	19	3.11	----	3.26	Silty CLAY to CLAY	,,
24.00	49.94	1.692	3.4	25	27	3.18	----	6.45	Clayey SILT to Silty CLAY	,,
24.50	41.30	1.605	3.9	21	22	3.24	----	5.29	,,	,,
25.00	27.01	1.656	6.1	27	28	3.31	----	3.38	CLAY	,,
25.50	30.20	1.910	6.3	30	32	3.38	----	3.80	,,	,,
26.00	46.64	2.473	5.3	47	48	3.45	----	5.99	,,	,,
26.50	48.29	1.918	4.0	24	25	3.51	----	6.20	Clayey SILT to Silty CLAY	,,
27.00	49.46	2.634	5.3	49	50	3.58	----	6.36	CLAY	,,
27.50	32.71	1.581	4.8	33	33	3.65	----	4.12	,,	,,
28.00	32.04	1.196	3.7	16	16	3.72	----	4.02	Clayey SILT to Silty CLAY	,,
28.50	34.73	1.180	3.4	17	17	3.78	----	4.38	,,	,,
29.00	31.67	1.044	3.3	16	16	3.85	----	3.97	,,	,,
29.50	30.81	1.216	3.9	21	21	3.92	----	3.85	Silty CLAY to CLAY	,,
30.00	28.66	1.110	3.9	19	19	3.99	----	3.56	,,	,,
30.50	27.01	0.920	3.4	14	13	4.05	----	3.33	Clayey SILT to Silty CLAY	,,
31.00	26.46	1.026	3.9	18	18	4.12	----	3.25	Silty CLAY to CLAY	,,
31.50	23.51	1.032	4.4	16	16	4.19	----	2.86	,,	,,
32.00	28.48	0.950	3.3	14	14	4.26	----	3.51	Clayey SILT to Silty CLAY	,,
32.50	26.76	0.953	3.6	13	13	4.32	----	3.28	,,	,,

John Sarmiento & Associates
Cone Penetration Testing Service

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214(LNY-73)

CPT NO.: CPT-2
DATE : 07-27-2000
Groundwater measured at 0.1 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
33.00	27.13	0.962	3.5	14	13	4.39	----	3.32	,,	,,
33.50	38.97	1.470	3.8	19	19	4.46	----	4.90	,,	,,
34.00	24.50	0.868	3.5	12	12	4.53	----	2.96	,,	,,
34.50	25.60	0.834	3.3	13	13	4.59	----	3.11	,,	,,
35.00	25.78	0.860	3.3	13	13	4.66	----	3.13	,,	,,
35.50	32.90	1.158	3.5	16	16	4.73	----	4.07	,,	,,
36.00	31.18	0.973	3.1	16	15	4.80	----	3.84	,,	,,
36.50	34.37	1.204	3.5	17	16	4.86	----	4.26	,,	,,
37.00	31.67	0.971	3.1	16	15	4.93	----	3.89	,,	,,
37.50	33.14	1.321	4.0	17	15	5.00	----	4.09	,,	,,
38.00	97.97	2.071	2.1	33	30	5.07	37	----	Silty SAND to Sandy SILT	,,
38.50	91.34	3.994	4.4	91	83	5.13	----	11.84	Very Stiff Fine Grained *	,,
39.00	83.92	2.095	2.5	28	25	5.20	36	----	Silty SAND to Sandy SILT	,,
39.50	59.82	3.268	5.5	60	53	5.27	----	7.62	Very Stiff Fine Grained *	,,
40.00	28.11	1.216	4.3	19	16	5.34	----	3.39	Silty CLAY to CLAY	,,
40.50	37.06	1.162	3.1	19	16	5.40	----	4.58	Clayey SILT to Silty CLAY	,,
41.00	31.18	0.895	2.9	16	13	5.47	----	3.79	,,	,,

DEPTH = Sampling interval (2 inches)
Qc = Tip bearing resistance
Fs = Sleeve friction resistance
Rf = Tip/Sleeve ratio
SPT = Equivalent Standard Penetration Test*
TotStr = Total Stress using est. density**
Phi = Soil friction angle***
Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf) (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)

References: * Robertson and Campanella, 1988
** Olsen, 1989 ***Durgunoglu & Mitchell, 1975

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214(LNY-73)

CPT NO.: CPT-3
DATE : 07-27-2000
Groundwater measured at 10.9 feet

Page 1 of 2

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0.50	136.44	1.220	0.9	27	44	0.06	43	----	SAND	120-130
1.00	21.07	1.240	5.9	21	34	0.13	----	2.80	CLAY	130-140
1.50	26.96	0.032	0.1	9	14	0.18	33	----	Silty SAND to Sandy SILT	110-120
2.00	30.28	1.114	3.7	15	24	0.25	----	4.02	Clayey SILT to Silty CLAY	130-140
2.50	30.35	1.075	3.5	15	24	0.31	----	4.03	''	''
3.00	32.26	1.198	3.7	16	26	0.38	----	4.28	''	''
3.50	26.90	1.094	4.1	18	29	0.45	----	3.56	Silty CLAY to CLAY	''
4.00	28.50	1.125	3.9	19	30	0.52	----	3.77	''	''
4.50	28.69	1.224	4.3	19	31	0.58	----	3.79	''	''
5.00	30.34	1.007	3.3	15	24	0.65	----	4.00	Clayey SILT to Silty CLAY	''
5.50	26.97	0.949	3.5	13	22	0.72	----	3.55	''	''
6.00	28.19	1.003	3.6	14	23	0.79	----	3.71	''	''
6.50	21.93	0.901	4.1	15	23	0.85	----	2.87	Silty CLAY to CLAY	''
7.00	20.88	1.052	5.0	21	31	0.92	----	2.72	CLAY	''
7.50	6.95	0.851	10.0	7	10	0.98	----	1.29	Organic Material	120-130
8.00	23.59	0.952	4.0	16	22	1.05	----	3.08	Silty CLAY to CLAY	130-140
8.50	27.09	1.064	3.9	18	24	1.12	----	3.54	''	''
9.00	25.86	1.009	3.9	17	23	1.18	----	3.37	''	''
9.50	28.57	1.062	3.7	14	18	1.25	----	3.73	Clayey SILT to Silty CLAY	''
10.00	22.91	1.012	4.4	15	19	1.32	----	2.97	Silty CLAY to CLAY	''
10.50	23.95	1.000	4.2	16	19	1.39	----	3.10	''	''
11.00	21.80	0.831	3.8	15	17	1.45	----	2.81	''	''
11.50	23.40	0.932	4.0	16	18	1.52	----	3.02	''	''
12.00	24.14	0.920	3.8	16	18	1.59	----	3.11	''	''
12.50	31.58	1.664	5.3	32	36	1.66	----	4.10	CLAY	''
13.00	25.44	1.262	5.0	25	29	1.72	----	3.28	''	''
13.50	22.85	1.090	4.8	23	25	1.79	----	2.93	''	''
14.00	28.93	1.162	4.0	19	21	1.86	----	3.73	Silty CLAY to CLAY	''
14.50	31.32	1.097	3.5	16	17	1.93	----	4.05	Clayey SILT to Silty CLAY	''
15.00	37.03	1.191	3.2	19	20	1.99	----	4.80	''	''
15.50	73.10	3.270	4.5	37	39	2.06	----	9.61	''	''
16.00	41.64	1.967	4.7	28	29	2.13	----	5.41	Silty CLAY to CLAY	''
16.50	32.37	1.306	4.0	22	22	2.20	----	4.17	''	''
17.00	34.16	1.129	3.3	17	18	2.26	----	4.40	Clayey SILT to Silty CLAY	''
17.50	39.74	1.200	3.0	16	16	2.33	----	5.14	Sandy SILT to Clayey SILT	''
18.00	42.57	1.256	3.0	17	17	2.40	----	5.52	''	''
18.50	41.04	1.142	2.8	16	17	2.47	----	5.31	''	''
19.00	38.08	1.059	2.8	15	15	2.53	----	4.91	''	''
19.50	35.51	1.170	3.3	18	18	2.60	----	4.56	Clayey SILT to Silty CLAY	''
20.00	36.31	1.130	3.1	18	18	2.67	----	4.66	''	''
20.50	37.35	1.211	3.2	19	19	2.74	----	4.80	''	''
21.00	42.38	1.255	3.0	17	17	2.80	----	5.46	Sandy SILT to Clayey SILT	''
21.50	41.41	1.346	3.3	21	21	2.87	----	5.33	Clayey SILT to Silty CLAY	''
22.00	35.87	1.171	3.3	18	18	2.94	----	4.59	''	''
22.50	28.50	1.036	3.6	14	14	3.01	----	3.60	''	''
23.00	27.21	0.951	3.5	14	14	3.07	----	3.42	''	''
23.50	35.87	1.277	3.6	18	18	3.14	----	4.57	''	''
24.00	31.69	1.501	4.7	21	21	3.21	----	4.01	Silty CLAY to CLAY	''
24.50	30.28	1.072	3.5	15	15	3.28	----	3.82	Clayey SILT to Silty CLAY	''
25.00	27.52	0.940	3.4	14	14	3.34	----	3.45	''	''
25.50	26.59	0.871	3.3	13	13	3.41	----	3.32	''	''
26.00	29.05	0.847	2.9	15	14	3.48	----	3.64	''	''
26.50	43.30	1.163	2.7	17	17	3.55	----	5.54	Sandy SILT to Clayey SILT	''
27.00	39.99	1.102	2.8	16	15	3.61	----	5.09	''	''
27.50	41.77	1.217	2.9	17	16	3.68	----	5.32	''	''
28.00	40.66	1.331	3.3	20	19	3.75	----	5.17	Clayey SILT to Silty CLAY	''
28.50	40.78	1.390	3.4	20	19	3.82	----	5.18	''	''
29.00	36.67	1.324	3.6	18	17	3.88	----	4.63	''	''
29.50	47.85	1.627	3.4	24	22	3.95	----	6.12	''	''
30.00	37.35	1.715	4.6	25	22	4.02	----	4.71	Silty CLAY to CLAY	''
30.50	41.64	1.699	4.1	21	18	4.09	----	5.28	Clayey SILT to Silty CLAY	''
31.00	40.41	1.640	4.1	20	17	4.15	----	5.11	''	''
31.50	40.66	1.683	4.1	20	17	4.22	----	5.14	''	''
32.00	46.62	1.459	3.1	19	16	4.29	----	5.93	Sandy SILT to Clayey SILT	''
32.50	51.22	2.031	4.0	26	21	4.36	----	6.54	Clayey SILT to Silty CLAY	''

John Sarmiento & Associates
Cone Penetration Testing Service

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214(LNY-73)

CPT NO.: CPT-3
DATE : 07-27-2000
Groundwater measured at 10.9 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
33.00	52.52	2.039	3.9	26	22	4.42	----	6.71	,,	,,
33.50	56.14	2.115	3.8	28	23	4.49	----	7.19	,,	,,
34.00	56.14	2.296	4.1	28	23	4.56	----	7.18	,,	,,
34.50	63.58	2.657	4.2	32	26	4.63	----	8.17	,,	,,
35.00	61.91	2.440	3.9	31	25	4.69	----	7.94	,,	,,
35.50	62.53	2.340	3.7	31	25	4.76	----	8.02	,,	,,
36.00	62.84	2.027	3.2	25	20	4.83	----	8.06	Sandy SILT to Clayey SILT	,,
36.50	65.11	2.171	3.3	26	21	4.90	----	8.35	,,	,,
37.00	60.68	2.542	4.2	30	24	4.96	----	7.76	Clayey SILT to Silty CLAY	,,
37.50	47.73	1.876	3.9	24	19	5.03	----	6.03	,,	,,
38.00	44.40	1.881	4.2	22	17	5.10	----	5.58	,,	,,
38.50	63.27	2.981	4.7	42	33	5.17	----	8.09	Silty CLAY to CLAY	,,
39.00	50.98	2.049	4.0	25	20	5.23	----	6.45	Clayey SILT to Silty CLAY	,,
39.50	51.28	2.075	4.0	26	20	5.30	----	6.48	,,	,,
40.00	43.42	1.400	3.2	22	17	5.37	----	5.43	,,	,,

DEPTH = Sampling interval (2 inches)
Qc = Tip bearing resistance
Fs = Sleeve friction resistance
Rf = Tip/Sleeve ratio
SPT = Equivalent Standard Penetration Test*
TotStr = Total Stress using est. density**
Phi = Soil friction angle***
Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf) (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)

References: * Robertson and Campanella, 1988
** Olsen, 1989 ***Durgunoglu & Mitchell, 1975

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214(LNY-73)

CPT NO.: CPT-4
DATE : 07-27-2000
Groundwater estimated at 10.0 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
0.50	2.38	0.072	3.0	2	4	0.06	----	0.47	CLAY	85-90
1.00	68.36	4.974	7.3	68	109	0.12	----	9.11	Very Stiff Fine Grained *	130-140
1.50	231.10	4.018	1.7	46	74	0.18	46	----	SAND	''
2.00	45.91	1.571	3.4	23	37	0.25	----	6.11	Clayey SILT to Silty CLAY	''
2.50	16.86	0.419	2.5	8	13	0.31	----	2.23	''	120-130
3.00	12.95	0.431	3.3	9	14	0.38	----	1.70	Silty CLAY to CLAY	''
3.50	14.16	0.601	4.2	14	23	0.44	----	1.86	CLAY	''
4.00	15.07	0.629	4.2	15	24	0.50	----	1.98	''	''
4.50	16.99	0.742	4.4	17	27	0.56	----	2.23	''	''
5.00	18.38	0.705	3.8	12	20	0.63	----	2.41	Silty CLAY to CLAY	''
5.50	23.88	0.670	2.8	12	19	0.69	----	3.14	Clayey SILT to Silty CLAY	''
6.00	24.18	0.841	3.5	12	19	0.75	----	3.17	''	130-140
6.50	27.87	1.012	3.6	14	22	0.82	----	3.66	''	''
7.00	25.22	1.086	4.3	17	26	0.89	----	3.30	Silty CLAY to CLAY	''
7.50	29.27	1.345	4.6	20	29	0.96	----	3.84	''	''
8.00	26.58	1.264	4.8	27	37	1.02	----	3.48	CLAY	''
8.50	25.16	1.242	4.9	25	34	1.09	----	3.28	''	''
9.00	22.71	1.425	6.3	23	30	1.16	----	2.95	''	''
9.50	23.87	1.197	5.0	24	31	1.22	----	3.10	''	''
10.00	37.87	1.585	4.2	25	32	1.29	----	4.96	Silty CLAY to CLAY	''
10.50	43.26	1.847	4.3	29	36	1.36	----	5.68	''	''
11.00	39.28	2.172	5.5	39	48	1.43	----	5.14	CLAY	''
11.50	49.71	2.025	4.1	25	30	1.49	----	6.53	Clayey SILT to Silty CLAY	''
12.00	44.47	2.104	4.7	30	35	1.56	----	5.82	Silty CLAY to CLAY	''
12.50	44.43	2.185	4.9	30	35	1.63	----	5.81	''	''
13.00	43.39	2.233	5.1	43	50	1.70	----	5.67	CLAY	''
13.50	33.69	1.623	4.8	34	38	1.76	----	4.37	''	''
14.00	31.18	1.544	5.0	31	35	1.83	----	4.03	''	''
14.50	28.84	1.514	5.3	29	32	1.90	----	3.72	''	''
15.00	40.69	2.117	5.2	41	45	1.97	----	5.29	''	''
15.50	42.84	1.974	4.6	29	31	2.03	----	5.58	Silty CLAY to CLAY	''
16.00	30.56	1.114	3.6	15	16	2.10	----	3.93	Clayey SILT to Silty CLAY	''
16.50	34.25	1.033	3.0	17	18	2.17	----	4.42	''	''
17.00	39.09	1.181	3.0	20	21	2.24	----	5.06	''	''
17.50	35.10	1.086	3.1	18	18	2.30	----	4.53	''	''
18.00	32.96	1.056	3.2	16	17	2.37	----	4.24	''	''
18.50	37.07	1.281	3.5	19	19	2.44	----	4.78	''	''
19.00	38.91	1.267	3.3	19	20	2.51	----	5.02	''	''
19.50	43.57	1.314	3.0	17	18	2.57	----	5.64	Sandy SILT to Clayey SILT	''
20.00	44.37	1.482	3.3	22	22	2.64	----	5.74	Clayey SILT to Silty CLAY	''
20.50	44.19	1.656	3.7	22	22	2.71	----	5.71	''	''
21.00	34.61	1.177	3.4	17	17	2.78	----	4.43	''	''
21.50	35.41	1.071	3.0	18	18	2.84	----	4.53	''	''
22.00	37.31	1.130	3.0	19	19	2.91	----	4.78	''	''
22.50	38.97	1.157	3.0	16	16	2.98	----	5.00	Sandy SILT to Clayey SILT	''
23.00	30.99	0.988	3.2	15	15	3.05	----	3.93	Clayey SILT to Silty CLAY	''
23.50	29.40	0.800	2.7	15	15	3.11	----	3.71	''	''
24.00	33.50	0.954	2.8	17	17	3.18	----	4.25	''	''
24.50	38.48	1.121	2.9	15	15	3.25	----	4.91	Sandy SILT to Clayey SILT	''
25.00	44.19	1.284	2.9	18	18	3.32	----	5.67	''	''
25.50	38.60	1.394	3.6	19	19	3.38	----	4.92	Clayey SILT to Silty CLAY	''
26.00	23.14	1.023	4.4	15	15	3.45	----	2.85	Silty CLAY to CLAY	''
26.50	34.05	1.242	3.6	17	17	3.52	----	4.31	Clayey SILT to Silty CLAY	''
27.00	35.47	1.368	3.9	18	18	3.59	----	4.49	''	''
27.50	34.67	1.423	4.1	23	23	3.65	----	4.38	Silty CLAY to CLAY	''
28.00	29.76	1.212	4.1	20	19	3.72	----	3.72	''	''
28.50	27.55	1.032	3.7	14	13	3.79	----	3.42	Clayey SILT to Silty CLAY	''
29.00	23.08	1.010	4.4	15	14	3.86	----	2.82	Silty CLAY to CLAY	''
29.50	31.29	1.184	3.8	16	15	3.92	----	3.91	Clayey SILT to Silty CLAY	''
30.00	39.77	1.510	3.8	20	18	3.99	----	5.04	''	''
30.50	39.09	1.272	3.3	20	18	4.06	----	4.94	''	''
31.00	35.71	1.287	3.6	18	16	4.13	----	4.49	''	''
31.50	50.13	1.600	3.2	20	18	4.19	----	6.40	Sandy SILT to Clayey SILT	''
32.00	32.58	1.141	3.5	16	14	4.26	----	4.06	Clayey SILT to Silty CLAY	''
32.50	38.04	1.588	4.2	25	22	4.33	----	4.78	Silty CLAY to CLAY	''

John Sarmiento & Associates
Cone Penetration Testing Service

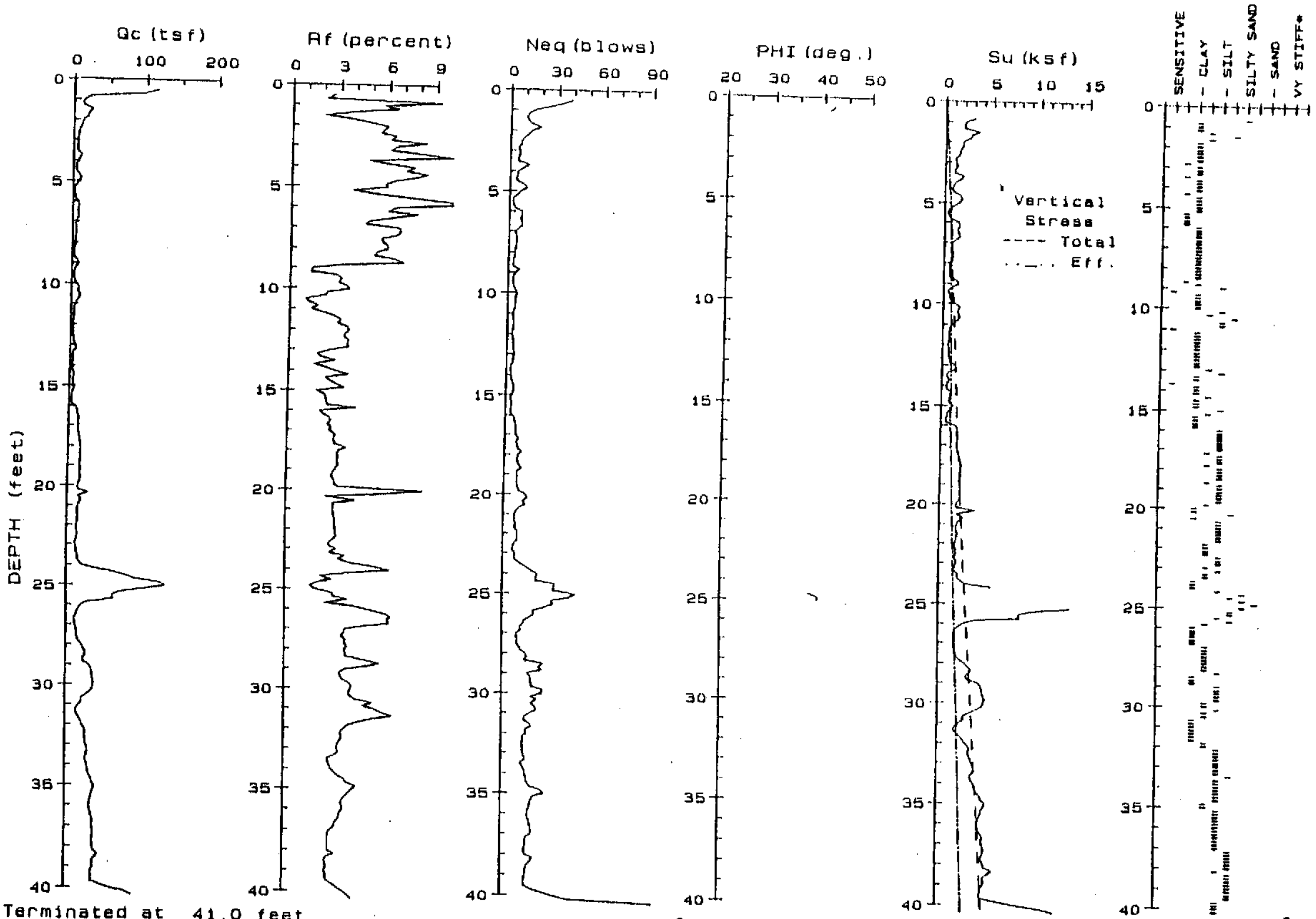
PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214(LNY-73)

CPT NO.: CPT-4
DATE : 07-27-2000
Groundwater estimated at 10.0 feet

DEPTH (feet)	Qc (tsf)	Fs (tsf)	Rf (%)	SPT (N)	SPT (N')	TotHzStr (ksf)	PHI (deg.)	SU (ksf)	SOIL BEHAVIOR TYPE	DENSITY RANGE (pcf)
33.00	65.96	3.590	5.4	66	56	4.40	----	8.50	Very Stiff Fine Grained *	,,
33.50	73.39	3.047	4.2	37	31	4.46	----	9.49	Clayey SILT to Silty CLAY	,,
34.00	41.12	1.810	4.4	27	23	4.53	----	5.18	Silty CLAY to CLAY	,,
34.50	40.00	1.499	3.7	20	16	4.60	----	5.03	Clayey SILT to Silty CLAY	,,
35.00	36.20	1.197	3.3	18	15	4.67	----	4.52	,,	,,
35.50	41.73	1.183	2.8	17	14	4.73	----	5.25	Sandy SILT to Clayey SILT	,,
36.00	49.21	1.482	3.0	20	16	4.80	----	6.24	,,	,,
36.50	42.64	1.354	3.2	21	17	4.87	----	5.36	Clayey SILT to Silty CLAY	,,
37.00	38.78	1.352	3.5	19	15	4.94	----	4.84	,,	,,
37.50	37.18	2.286	6.1	37	30	5.00	----	4.62	CLAY	,,
38.00	63.15	3.855	6.1	63	50	5.07	----	8.08	Very Stiff Fine Grained *	,,
38.50	66.88	3.202	4.8	67	53	5.14	----	8.58	,,	,,
39.00	44.60	2.055	4.6	30	23	5.21	----	5.60	Silty CLAY to CLAY	,,
39.50	41.49	1.466	3.5	21	16	5.27	----	5.18	Clayey SILT to Silty CLAY	,,
40.00	55.60	1.866	3.4	28	22	5.34	----	7.06	,,	,,
40.50	48.67	1.625	3.3	24	19	5.41	----	6.13	,,	,,
41.00	50.80	1.365	2.7	20	16	5.48	----	6.41	Sandy SILT to Clayey SILT	,,

DEPTH = Sampling interval (2 inches)
Qc = Tip bearing resistance
Fs = Sleeve friction resistance
Rf = Tip/Sleeve ratio
SPT = Equivalent Standard Penetration Test*
TotStr = Total Stress using est. density**
Phi = Soil friction angle***
Su = Undrained Soil Strength* (Nk=10 for Qc<9 tsf) (Nk=12 for Qc=9 to 12 tsf) (Nk=15 for Qc>12 tsf)

References: * Robertson and Campanella, 1988
** Olsen, 1989 ***Durgunoglu & Mitchell, 1975



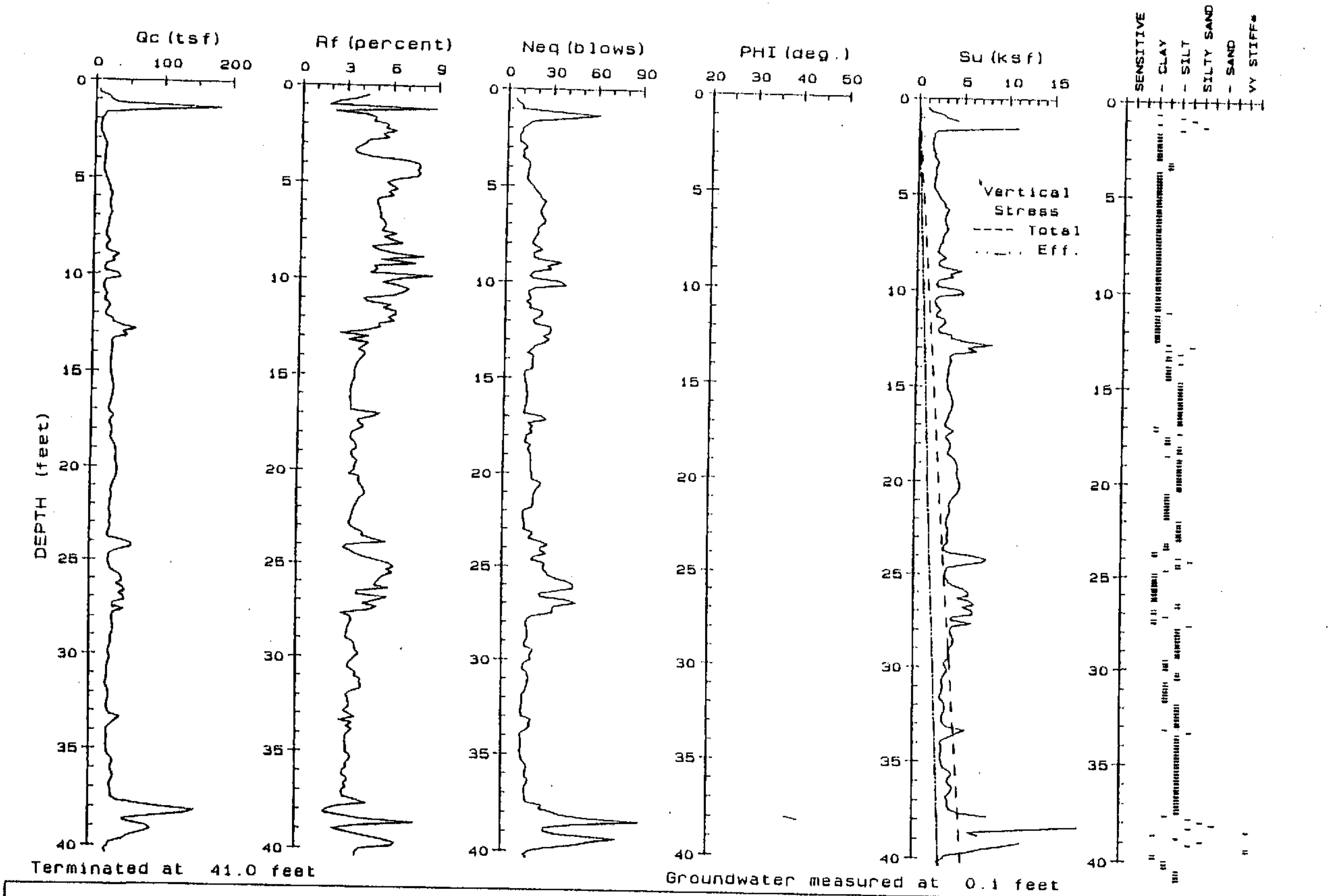
Terminated at 41.0 feet

Groundwater measured at 5.9 feet

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214 (LNY-73)

CPT NO.: CPT-1
DATE : 07-27-2000

John Sarmiento & Associates
Cone Penetration Testing Service



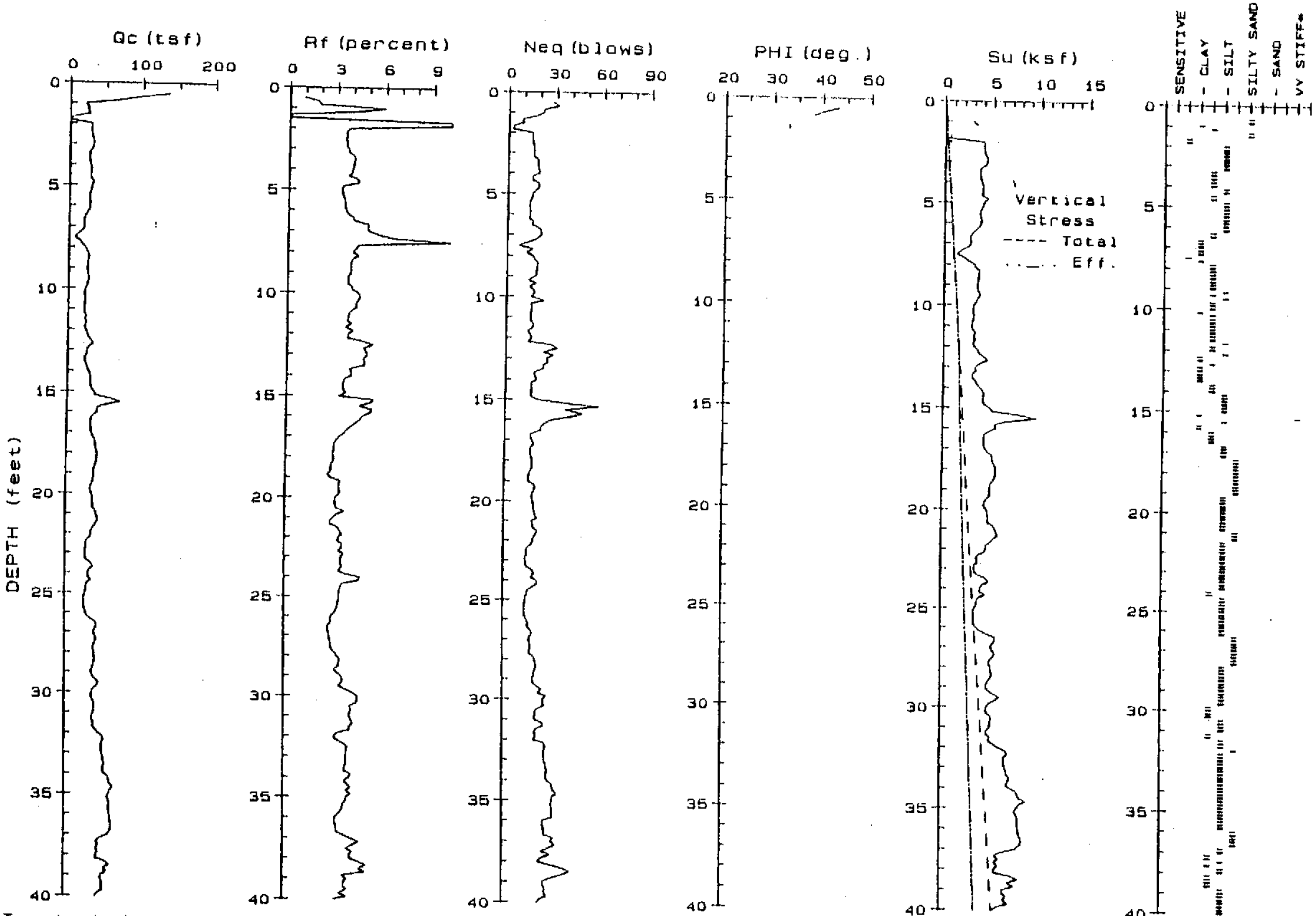
Terminated at 41.0 feet

Groundwater measured at 0.1 feet

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214 (LNY-73)

CPT NO.: CPT-2
DATE : 07-27-2000

John Sarmiento & Associates
Cone Penetration Testing Service



Terminated at 40.0 feet

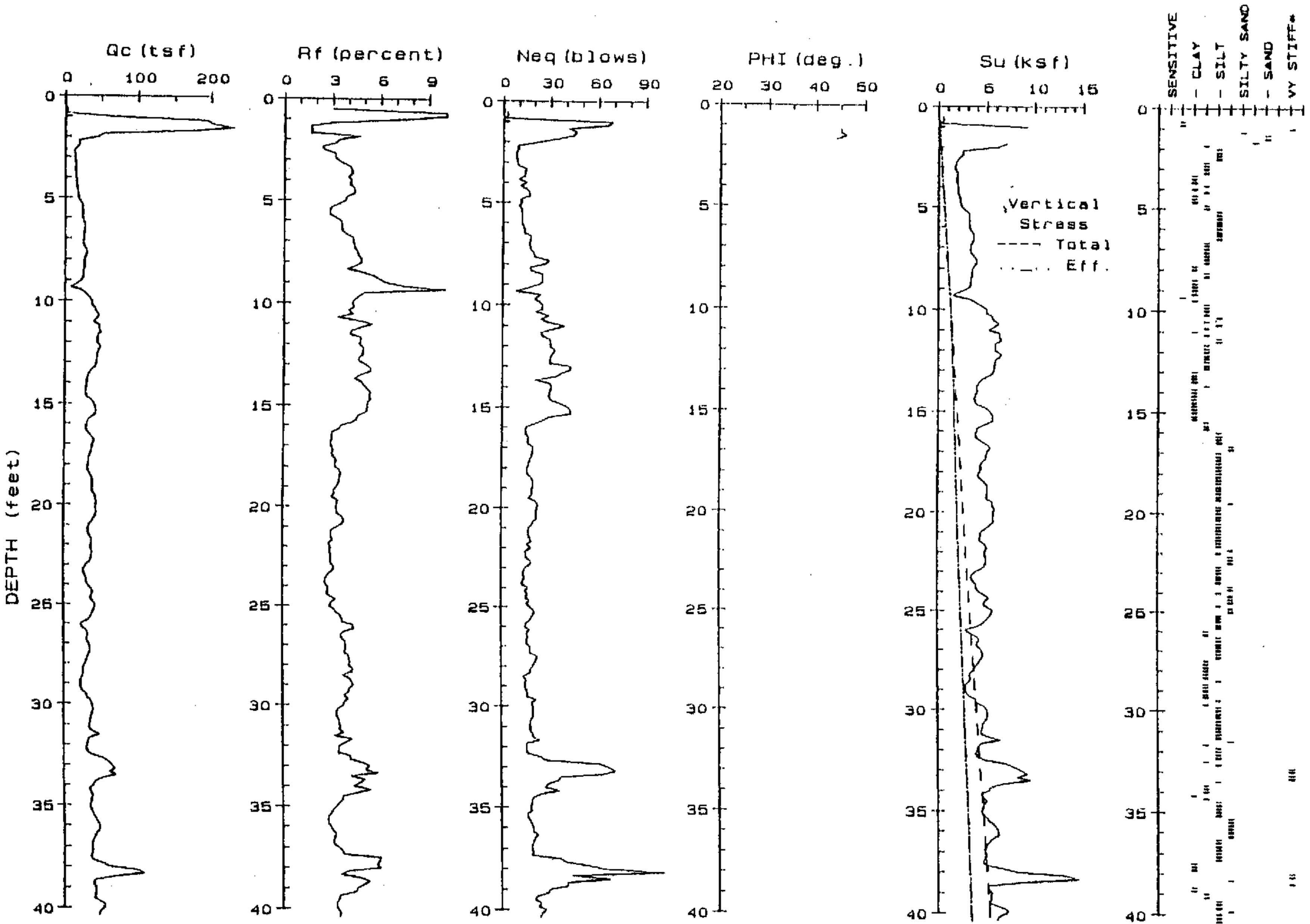
Groundwater measured at 10.9 feet

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214 (LNY-73)

CPT NO.: CPT-3
DATE : 07-27-2000

John Sarmiento & Associates

Cone Penetration Testing Service



Terminated at 41.0 feet

Groundwater estimated at 10.0 feet

PROJECT: 230 BAY PLACE
LOCATION: Oakland CA
PROJ. NO.: P10214 (LNY-73)

CPT NO.: CPT-4
DATE : 07-27-2000

John Sarmiento & Associates

Cone Penetration Testing Service

Avalon Bay Rear Slope

PROJECT IDENTIFICATION

Title:	Avalon Bay Rear Slope
Project Number:	201071
Designer:	Dan Dyckman

Description:

Steep rear slope to be cut for new retaining wall

File path and name: Not saved, yet
Date and time of creating the input data file: Wed May 23 14:39:50 2001

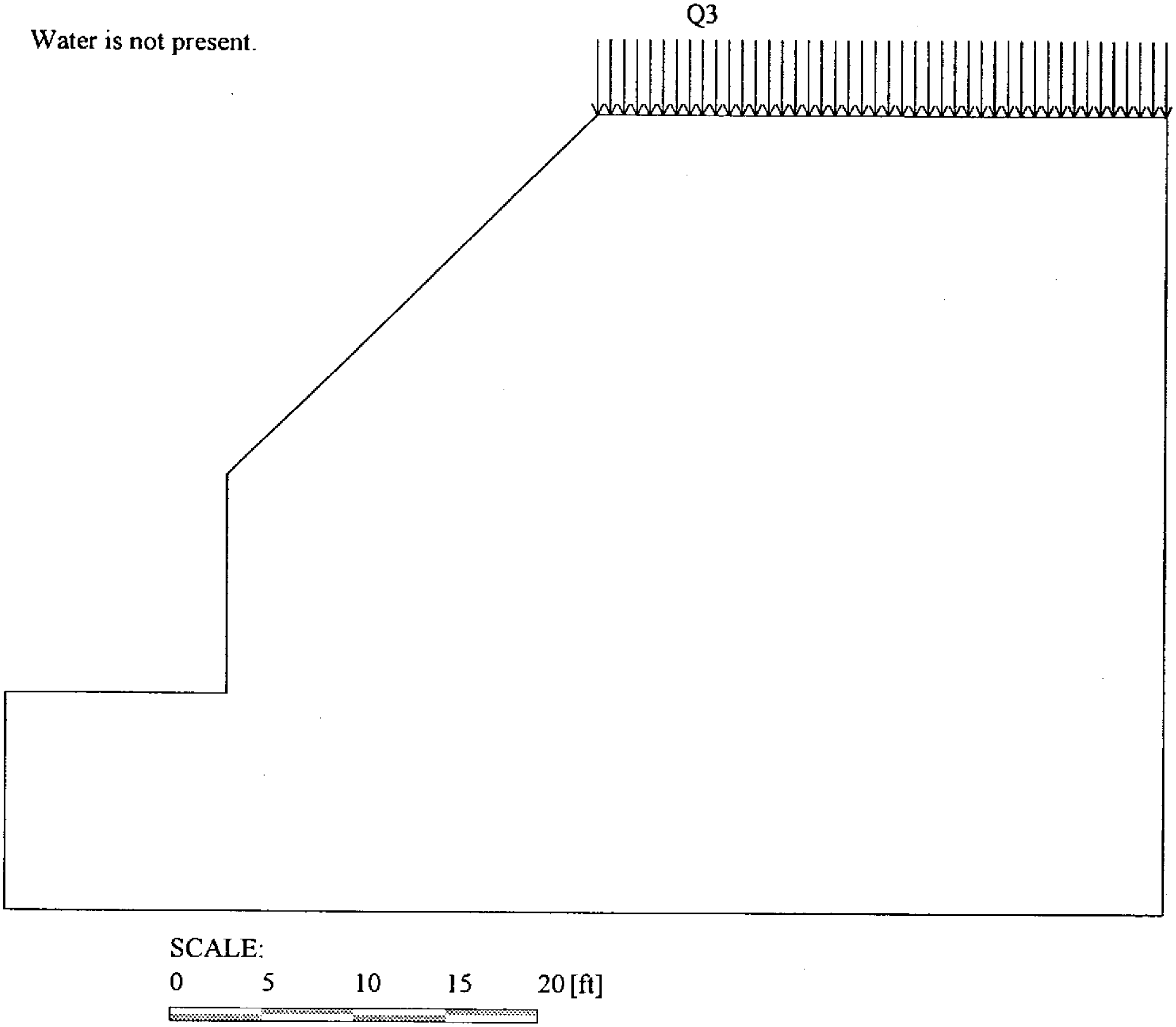
Design Philosophy and Program Developed by:

Dov Leshchinsky, Ph.D.
33 The Horseshoe
Newark, Delaware 19711, USA

GEOMETRY AND LOADING DATA

Height of slope, H [ft]	12.00
Slope angle, i °	90.00
Horizontal length, A [ft]	0.00
Horizontal length, B [ft]	20.00
Backslope angle, β °	45.00
Slope at bottom of wall, α °	0.00
Surcharge load over A, Q1 [lb/ft ²]	0.00
Surcharge load over backslope B, Q2 [lb/ft ²]	0.00
Surcharge load away from backslope, Q3 [lb/ft ²]	200.00

Water is not present.



SOIL DATA

REINFORCED SOIL:	Internal angle of friction, $\phi°$	38.0
	Cohesion, c [lb/ft ^{2}]	600.00
	Moist unit weight, γ [lb/ft ^{3}]	125.00
BACKFILL SOIL:	Internal angle of friction, $\phi°$	38.0
	Cohesion, c [lb/ft ^{2}]	600.00
	Moist unit weight, γ [lb/ft ^{3}]	125.00
FOUNDATION SOIL:	Internal angle of friction, $\phi°$	35.0
	Cohesion, c [lb/ft ^{2}]	1000.00
	Moist unit weight, γ [lb/ft ^{3}]	125.00

GENERAL DATA

Assumed angle of interwedge force (direct sliding analysis), δ °	20.00
Pullout interaction coefficient (reinforced soil), Ci	0.90
Pullout interaction coefficient (foundation soil), Ci	0.90
Direct sliding coefficient (along reinforced soil), Cds	1.00
Direct sliding coefficient (along foundation soil), Cds	1.00

Uniform length of all layers was specified.

SEISMIC PARAMETERS

Horizontal seismic coefficient, Kh	0.00
Vertical seismic coefficient, Kv	0.00

Kh and Kv ARE being applied to the reinforced mass and surcharge in direct sliding analysis.

FOUNDATION EFFECTS

Slip surfaces in tieback and compound analyses are allowed to penetrate the foundation soil.

Bishop's deepseated analysis was invoked and circles may penetrate the foundation to a maximum depth of [ft]	39.37

GEOSYNTHETIC DESIGN PARAMETERS

(Optimized spacing was conducted by ReSlope, where strength = 1.00 [lb/ft],
Dmax = 5.00, Dmin = 1.00, Dbottom = 0.00 [ft]).

Reduction factor for installation damage, RFid	1.20
Reduction factor for durability, RFd	1.10
Reduction factor for creep, RFc	2.50
Coverage ratio, Rc	1.00

SPECIFIED FORCE ORIENTATION

Relative orientation of reinforcement is prescribed, ROR	0.00

GENERAL SAFETY FACTORS

Factor of safety on soil shear strength	1.30
Factor of safety on geosynthetic strength	1.30
Factor of safety on pullout resistance	1.50
Factor of safety on direct sliding resistance	1.10

SUMMARY OF TIEBACK AND COMPOUND RESULTS

#	Elevation [ft]	Length [ft]	Mode of Failure	Strength: Required, Tr [lb/ft]	Strength: Ultimate, T-ult [lb/ft]	Strength: Long-term (design) T-ltds [lb/ft]	Actual Overall Fs	Status
1	0.00	0.00	Tieback	0.00	1.00	0.30	44225733.45	OK
2	2.00	0.00	Tieback	0.00	1.00	0.30	44225733.45	OK
3	7.00	0.00	Tieback	0.00	1.00	0.30	44225733.45	OK

DETAILED RESULTS OF TIEBACK AND COMPOUND ANALYSES

#	Elevation [ft]	Total Length [ft]	Embedded Length to resist pullout, Le [ft]	Length to slip surface, La [ft]	Strength for: Compound stability (available) T-compound [lb/ft]	Strength for: Tieback (required) T-tieback [lb/ft]	Controlling Mode of Failure
1	0.00	0.00	0.00	0.00	0.23	0.00	Tieback
2	2.00	0.00	0.00	0.00	0.23	0.00	Tieback
3	7.00	0.00	0.00	0.00	0.23	0.00	Tieback

RESULTS OF DIRECT SLIDING AND DEEPSEATED ANALYSES

DIRECT SLIDING

Required length of bottom layer to produce the specified Fs-direct sliding = 1.10 is 0.00 ft.
Maximum length based on compound and tieback analyses to insure Fs-uncertainties = 1.30 and Fs-pullout = 1.50, is 0.00 ft.

DEEPSEATED

Deepseated factor of safety, Fs-deepseated, based on Bishop's analysis, is 2.27.

The critical circle is forced to pass outside the reinforced zone defined by the bottom geosynthetic layer; its maximum potential depth is restricted to 39.37 ft.
The critical circle is at: Xc = 0.00, Yc = 12.00, Radius = 12.00 feet.
In case the crest elevation is above H, ReSlope assumes a tension crack between the crest and H (see graphic screen).

NOTE: To obtain satisfactory Fs-deepseated, re-run ReSlope with a larger specified value of Fs-direct sliding. This will force deeper circles that should yield larger deepseated safety factor.

TIEBACK & COMPOUND

Tieback/compound slip surfaces are not restricted from penetrating the foundation soil.

Avalon Bay Rear Slope

PROJECT IDENTIFICATION

Title:	Avalon Bay Rear Slope
Project Number:	201071
Designer:	Dan Dyckman

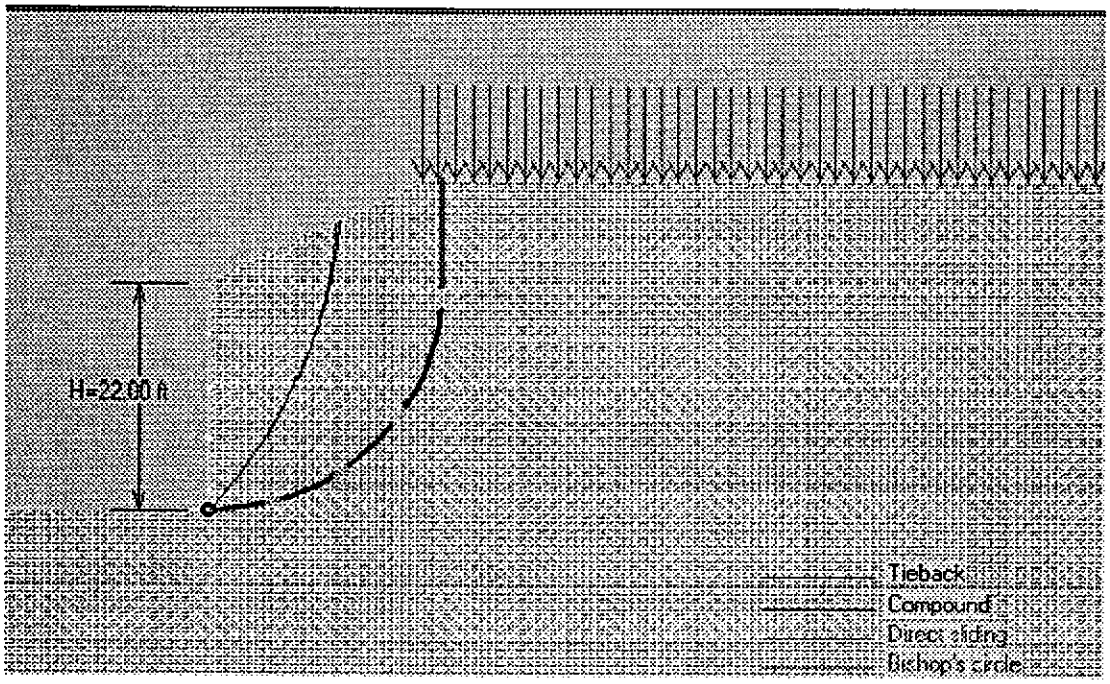
Description:

Steep rear slope to be cut for new retaining wall

File path and name: Not saved, yet
Date and time of creating the input data file: Wed May 23 14:39:50 2001

Design Philosophy and Program Developed by:

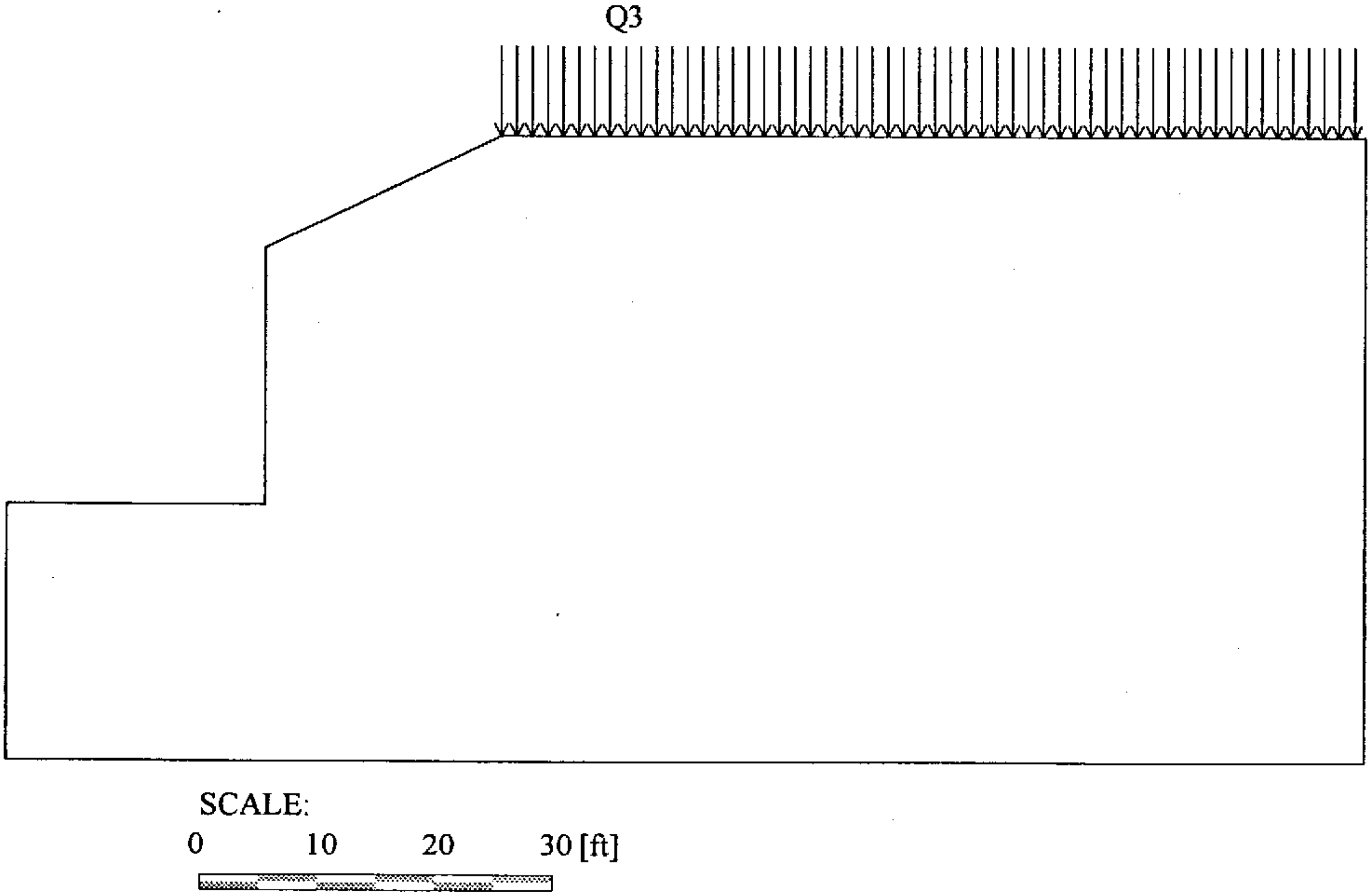
Dov Leshchinsky, Ph.D.
33 The Horseshoe
Newark, Delaware 19711, USA


H=22.00 ft
Tieback
Compound
Direct sliding
Bishop's circle

GEOMETRY AND LOADING DATA

Height of slope, H [ft]	22.00
Slope angle, i °	90.00
Horizontal length, A [ft]	0.00
Horizontal length, B [ft]	20.00
Backslope angle, β °	26.00
Slope at bottom of wall, α °	0.00
Surcharge load over A, Q1 [lb/ft ^{2}]	0.00
Surcharge load over backslope B, Q2 [lb/ft ^{2}]	0.00
Surcharge load away from backslope, Q3 [lb/ft ^{2}]	200.00

Water is not present.



SOIL DATA

REINFORCED SOIL:	Internal angle of friction, φ°	38.0
	Cohesion, c [lb/ft ²]	600.00
	Moist unit weight, γ [lb/ft ³]	125.00
BACKFILL SOIL:	Internal angle of friction, φ°	38.0
	Cohesion, c [lb/ft ²]	600.00
	Moist unit weight, γ [lb/ft ³]	125.00
FOUNDATION SOIL:	Internal angle of friction, φ°	35.0
	Cohesion, c [lb/ft ²]	1000.00
	Moist unit weight, γ [lb/ft ³]	125.00

GENERAL DATA

Assumed angle of interwedge force (direct sliding analysis), δ °	20.00
Pullout interaction coefficient (reinforced soil), Ci	0.90
Pullout interaction coefficient (foundation soil), Ci	0.90
Direct sliding coefficient (along reinforced soil), Cds	1.00
Direct sliding coefficient (along foundation soil), Cds	1.00

Uniform length of all layers was specified.

SEISMIC PARAMETERS

Horizontal seismic coefficient, Kh	0.00
Vertical seismic coefficient, Kv	0.00

Kh and Kv ARE being applied to the reinforced mass and surcharge in direct sliding analysis.

FOUNDATION EFFECTS

Slip surfaces in tieback and compound analyses are allowed to penetrate the foundation soil.

Bishop's deepseated analysis was invoked and circles may penetrate the foundation to a maximum depth of [ft] 39.37

GEOSYNTHETIC DESIGN PARAMETERS

(Optimized spacing was conducted by ReSlope, where strength = 1.00 [lb/ft], Dmax = 5.00, Dmin = 1.00, Dbottom = 0.00 [ft]).

Reduction factor for installation damage, RFid	1.20
Reduction factor for durability, RFd	1.10
Reduction factor for creep, RFc	2.50
Coverage ratio, Rc	1.00

SPECIFIED FORCE ORIENTATION

Relative orientation of reinforcement is prescribed, ROR 0.00

GENERAL SAFETY FACTORS

Factor of safety on soil shear strength	1.30
Factor of safety on geosynthetic strength	1.30
Factor of safety on pullout resistance	1.50
Factor of safety on direct sliding resistance	1.10

RESULTS OF DIRECT SLIDING AND DEEPSEATED ANALYSES

DIRECT SLIDING

Required length of bottom layer to produce the
specified Fs-direct sliding = 1.10 is 0.00 ft.
Maximum length based on compound and tieback analyses to insure
Fs-uncertainties = 1.30 and Fs-pullout = 1.50, is 0.00 ft.

DEEPSEATED

Deepseated factor of safety, Fs-deepseated, based on Bishop's analysis, is 2.15.

The critical circle is forced to pass outside the reinforced zone defined by the bottom geosynthetic layer; its maximum potential depth is restricted to 39.37 ft.
The critical circle is at: Xc = 0.00, Yc = 22.00, Radius = 22.00 feet.
In case the crest elevation is above H, ReSlope assumes a tension crack between the crest and H (see graphic screen).

NOTE: To obtain satisfactory Fs-deepseated, re-run ReSlope with a larger specified value of Fs-direct sliding. This will force deeper circles that should yield larger deepseated safety factor.

TIEBACK & COMPOUND

Tieback/compound slip surfaces are not restricted from penetrating the foundation soil.