GEOTECHNICAL ENGINEERING INVESTIGATION REPORT

OCTOBER 18, 2007 PROJECT NO: 4972

PROJECT: PROPOSED FIVE STORY STRUCTURE WITH A ONE LEVEL SUBTERRANEAN PARKING GARAGE

LOCATION: 3635 ELM AVENUE, CITY OF LONG BEACH, CALIFORNIA

CLIENT: DIDM DEVELOPMENT CORPORATION

CALWEST GEOTECHNICAL
CONSULTING ENGINEERS

889 PIERCE COURT, SUITE 101
THOUSAND OAKS, CA 91360
818-991-7148 • 805-497-1244

October 18, 2007 Project No. 4972

DIDM Development Corporation
P.O. Box 9148
Calabasas, California 91372

SUBJECT: **GEOTECHNICAL ENGINEERING INVESTIGATION, PROPOSED FIVE STORY STRUCTURE WITH A ONE LEVEL SUBTERRANEAN PARKING GARAGE, 3635 ELM AVENUE, CITY OF LONG BEACH, CALIFORNIA.**

REFERENCES: TOPOGRAPHIC SURVEY, A.P.N. 7145-007-022, 3635 ELM AVENUE, LONG BEACH, CALIFORNIA, PREPARED BY LARRY PEARSON LAND SURVEYOR, WORK ORDER 07-3649, DATED SEPTEMBER 2007.

INTRODUCTION

This Geotechnical Engineering Report presents the results of our geotechnical engineering investigation performed for the proposed five story structure with a one level subterranean parking garage, located at 3635 Elm Avenue, City of Long Beach, California. The purpose of our investigation has been to describe the geotechnical conditions at the subject site and evaluate how those conditions may affect the proposed development.

The following report describes our scope of work and presents our professional opinions regarding the proposed development, in the form of findings, conclusions and geotechnical recommendations. The Location Map in Appendix A shows the approximate location of the subject site land surrounding vicinity.

SCOPE OF WORK

Our investigation has been directed at identification and evaluation of geotechnical conditions at the subject site that may impact the proposed development. Our investigation was conducted during September and October 2007 and included the following tasks:

- Consultation with the client during the field subsurface investigation phase and subsequent report preparation;
- Review of published geotechnical information relevant to the site and surrounding areas, available in our files;
- Performed a site reconnaissance to assess surficial conditions at the subject site;
- Excavation, sampling, and logging of two exploratory test borings. The exploratory test borings were excavated to a maximum depth of approximately 50 feet below the existing grade and were backfilled with the excavated material at the completion of the subsurface investigation phase. The logs of the exploratory test borings are included in Appendix B;

- Sampling, utilizing a 2-1/2 inch diameter ring sampler (ASTM D-3550 shoe, similar to ASTM d-1586). The samples were obtained by driving the sample with a 140 pound hammer, dropping 30 inches, in accordance with ASTM D-1586;
- Perform Standard Penotrometer Tests (SPT), in accordance with ASTM D-1586;
- Laboratory testing and analyses of selected samples to measure their pertinent index and engineering properties. Laboratory testing procedures and results of the laboratory tests are included in Appendix C;
- Preparation of a Geotechnical Map, utilizing as a basis, the referenced topographic survey provided by the client. The Geotechnical Map is included in Appendix D. We make no representations regarding the accuracy of the supplied map;
- Preparation of retaining wall design analyses and foundation bearing capacity analyses, utilizing the laboratory test results described above. The retaining wall design analyses and foundation bearing capacity analyses are included in Appendix E;
- Review and geotechnical engineering analysis of the available geotechnical data and our laboratory test results described above;
- Preparation of this formal report presenting our professional opinions regarding the proposed development, in the form of findings, conclusions, and geotechnical recommendations.

PROPOSED DEVELOPMENT

Information concerning the proposed development was provided by the client. It is our understanding the proposed development will consist of the construction of a five story building with a one level subterranean parking garage. It is anticipated that the above grade construction will be supported above the concrete and masonry subterranean parking garage. It is further anticipated the subterranean parking garage will be supported above conventional continuous footings, pad footings, and grade beams extending into the dense natural soil exposed in the excavation. Braced retaining walls will be utilized for the perimeter of the subterranean parking garage.

Grading associated with the proposed development will consist of temporary and permanent excavations for the subterranean parking garage, with subsequent backfilling of the temporary excavation area to a certified compacted fill. Specific development plans have not been provided and await, in part, the recommendations of this report.

SITE DESCRIPTION

The subject property is located at 3635 Elm Avenue, City of Long Beach, California. The property is legally identified as APN 7145-007-022. The property is currently vacant with an asphalt parking lot at the westerly portion of the site. The property is located at the south west corner of Elm Avenue and 37th Street. The subject site is relatively level with minor elevation differential from west to east. Commercial structures are located to the west of the site and Temple Beth Shalom is located to the south of the property.

SUBSURFACE CONDITIONS

Subsurface conditions beneath the subject site have been interpreted and characterized based on our observations during the site reconnaissance and in the two exploratory test borings, which were excavated to a maximum depth of approximately 50 feet below the existing grade. Earth materials observed during our investigation consist of dense natural terrace deposits.

The descriptions provided below pertain only to subsurface conditions revealed at the time of our field exploration in September 2007. Certain subsurface conditions, such as groundwater levels and the consistency of near-surface soils will vary with the seasons. The logs of the exploratory test borings are included in Appendix B.

Soil

Natural terrace deposits overly the subject site. The terrace deposits consists of a silty sand and clayey sand, generally described as yellowish orange brown to grayish brown slightly moist to moist and is dense to very dense with depth.

Groundwater

Groundwater was not encountered at the maximum depth of the field subsurface exploration, approximately 50 feet below the existing grade. However, it must be noted that fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the subsurface exploration and available recorded data.

SEISMIC CONSIDERATIONS

The subject site is not located within any California Special Studies Zone, and no known active faults cross the site or are in close proximity. The site, however, as all the Southern California area, is located in a seismically active region and will be subject to moderate to strong ground shaking should any of the many active Southern California faults produce an earthquake. Potential hazards from earthquakes in the vicinity of the site, aside from strong ground shaking, may include fault rupture, landslides, seiches, tsunamis, lurching, liquefaction, and seismically induced settlement.

Earthquakes are generally characterized by magnitude, which is a quantitative measure of the strength of the earthquake, based on the strain energy released during a seismic event. The magnitude, which is often quantified on the Richter scale, is independent of the site in question. The intensity of an earthquake at any given site, however, is affected by the magnitude, distance between the site and the hypocenter, and the geologic conditions between the site and hypocenter. Intensity is often measured, utilizing the MERCALLI scale. Intensity is generally greater in areas underlain by unconsolidated soils, rather than areas underlain by bedrock.

Two terms used to describe earthquakes are "maximum credible" and maximum probable." A "maximum credible" earthquake (MCE) refers to the maximum earthquake capable of occurring under the presently known teutonic framework. The "maximum probable" earthquake (MPE) refers to the maximum earthquake likely to occur during a 100-year event. The MPE is sometimes considered the design earthquake used to design earthquake-resistant structures.

The California Building Code (CBC) is often followed in seismic structural design. The CBC requirements are based on ground motions with 10% exceeding the 50-year, which correspond to a return period of approximately 475 years. Structural design criteria are provided for design based on the CBC static force procedure, for the 2001 edition of the California Building Code. The CBC site characteristics are listed under the following "Ground Shaking" section.

Fault Rupture

An earthquake is caused when strained energy and rocks are suddenly released by movement along a plane of weakness. Occasionally, fault movement propagates upward through the subsurface materials and causes displacement of the ground surface. Surface rupture usually occurs along the traces of known active or potentially active faults, although many historic events have occurred on faults not previously known to be active. Since no active faults are known to cross the property, the risk of earthquake-induced ground rupture occurring across the subject site appears to be remote.

Ground Shaking

Should a major earthquake occur with an epicenter location close to the subject site, ground shaking at the site will undoubtedly be severe, as it will be for other properties in the general vicinity. Lateral forces due to earthquake loading may be calculated utilizing the formulas presented in the 2001 edition of the California Building Code.

2001 California Building Code: Structural design criteria, based on the 2001 California Building Code Static Force Procedure, should incorporate the following site characteristics:

TABLE NO.	CHARACTERISTICS	FACTOR
16-I	Seismic Zone Factor	0.4
16-J	Soil Profile Type	S_d
16-Q	Seismic coefficient	Ca=0.44 Na
16-R	Seismic Coefficient	Cv=0.64 Nv
16-S	Near-source Factor	Na=1.3
16-T	Near-source Factor	Nv=1.6

Conformance with the above listed criteria for seismic design does not constitute any kind of warranty, guarantee, or assurance that significant structural damage or ground failure will not occur if a maximum level earthquake occurs. The primary goal of seismic design is to protect life and limb, and catastrophic failure, and not to avoid all damage, since such design may be economically prohibitive.

Liquefaction Potential

According to the State of California Seismic Hazard Zone Map included in Appendix A, the subject property is not located in an area subject to liquefaction. Many factors influence a soil potential for liquefaction during an earthquake. These factors include magnitude and proximity of an earthquake, duration of shaking, soil types, grain size distribution, clay fraction content, density, angularity, effective over-burden, location of groundwater table, soils transmisivity, as well as many others. There must be four conditions for soil liquefaction to occur:

1. The soil must be completely saturated and the groundwater table within 40 feet of the surface.
2. The soil must be in a loose state (non-cemented) and consist mostly of silts and fine sands. The relative density of the soil must be less than 70 to 75 percent. Coarse sands and gravels may liquefy if subjected to extreme intense shocks.
3. The soils are poorly graded, with a predominant grain size between 0.5 and 0.10mm with an approximate uniformity coefficient of less than 10.
4. The soil must be subjected to prolonged shocks to compress the saturated material that may be provided by an earthquake.
5. The pore water pressure built up during the shock must exceed the inner-granular pressure within the soil mass, which is based on over-burdened soil weight.

Factors that would eliminate or minimize the potential for liquefaction are:

A. A clay content (determined by grain size smaller than 0.005mm) greater than 20%.

B. Absence of groundwater within 40 feet of the ground surface.

C. Relative density of underlying soils greater than 70 to 75 percent.

Under the influence of severe ground shaking, the soil underlying the subject site proposed for development, based upon the known soil consistency and depth to groundwater, is not considered subject to liquefaction.

LABORATORY TESTING AND ANALYSES

Laboratory tests were performed on bulk and relatively undisturbed ring samples considered representative of the earth materials encountered during our subsurface exploration. These tests were performed to measure the pertinent index and engineering properties of the underlying earth materials. After a visual classification in the field, samples were returned to the laboratory, classifications were checked, and a testing program was established. Laboratory testing included classification of the soils in accordance with the Unified Soil Classification System.

Insitu moisture content and dry weight for samples were developed in accordance with ASTM D-2937. Shear strength characteristics were assessed from results of direct shear tests on relative, undisturbed samples. Classification tests consist of maximum density-optimum moisture content, per ASTM D-1557, grain size analysis for ASTM D-422, and expansion index per ASTM Standard 4829. A complete explanation of the laboratory testing procedures along with the laboratory test results, are included in Appendix C.

FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

CalWest Geotechnical has performed a geotechnical engineering investigation for the proposed five story structure with a one level subterranean parking garage, located at 3635 Elm Avenue, City of Long Beach, California Based upon our investigation, corresponding geotechnical analyses, and experience with similar projects, the proposed development is considered feasible from a geotechnical engineering standpoint, provided our recommendations are made part of the development plans and implemented during construction.

As previously stated, the proposed development will consist of the construction of a five story building with a one level subterranean parking garage. It is anticipated that the above grade construction will be supported above the concrete and masonry subterranean parking garage. It is further anticipated the subterranean parking garage will be supported above conventional continuous footings, pad footings, and grade beams extending into the dense natural soil exposed in the excavation. Braced retaining walls will be utilized for the perimeter of the subterranean parking garage.

Grading associated with the proposed development will consist of temporary and permanent excavations for the subterranean parking garage, with subsequent backfilling of the temporary excavation area to a certified compacted fill. Specific development plans have not been provided and await, in part, the recommendations of this report.

The recommendations that follow are presented as guidelines to be utilized during the design and construction of the proposed project, and have been prepared with the understanding that CalWest Geotechnical will be given the opportunity to review the site development plans prior to construction and will observe, test and advise during site grading and foundation construction. Prior to construction, it is recommended that a preconstruction meeting be held with the project engineering consultants, owner and general contractor to review the plans and specifications, and discuss scheduling of the project.

SITE PREPARATION, GRADING, COMPACTION AND UTILITY TRENCH BACKFILL

Site preparation and grading should be performed in compliance with all applicable grading codes and the minimum specifications outlined below. In-grading observations and testing will be necessary during all phases of project construction to allow CalWest Geotechnical to provide certification of the finished project.

Ground Preparation

A. All areas to be backfilled with compacted fill should be cleaned of loose debris and excavated to expose the dense natural soil. The excavations should be observed and approved by the project geotechnical consultant in the field prior to placing controlled compacted fill.

B. The soil surface exposed by stripping and excavation activities should be scarified to a minimum depth of eight inches, moisture conditioned to produce a soil-water content of about two percent above optimum moisture content and compacted to a minimum 90 percent relative compaction, based on ASTM Test D1557.

Fill Placement

A. Certified compacted fill may be placed to design grades using onsite inorganic soils or approved import.

B. Soil proposed for use as structural fill should be inorganic, free from deleterious materials, and contain no more than 15 percent by weight of rocks larger than four inches (largest dimension) and no rocks larger than six inches. Irreducible rock that exceeds a maximum dimension of six inches shall not be placed in the upper ten feet of any certified compacted fill.

We expect that materials excavated onsite will be suitable for use as certified compacted fill provided they do not contain appreciable quantities of organic debris.

Where in-place moisture content exceeds optimum values, the materials may need to be spread and dried, or mixed with dryer material. Final determination will be provided in the field by the project geotechnical consultants at the time the excavations take place.

Excavated material containing excessive organic debris will not be suitable for use in the certified compacted fill. Materials deemed unsuitable should be wasted offsite or as designated by the project architect or geotechnical consultant.

C. The approved material should be placed in layers, each not exceeding eight inches in thickness (before compaction), water conditions to about two percent above optimum moisture content and compacted to a minimum 90 percent relative compaction based on ASTM Test D1557.

Where cohesive soil, having less than 15 percent finer than 0.005 mm, is used for the certified compacted fill, it shall be compacted to a minimum **95** percent relative compaction, based on ASTM Test D1557.

D. Fill compaction tests should be performed during placement of the future fills to verify acceptable compaction and moisture content. At a minimum, one test should be performed within each 12 to 24 inches (vertical depth) or 1000 cubic yards of fill (whichever is less). More frequent testing may be required by the geotechnical consultant.

E. The upper 12 inches of pavement subgrade should be compacted to a minimum relative compaction of 95 percent.

F. If construction takes place during the winter months or unseasonable rainy periods, additional winterizing and erosion-control recommendations may be necessary.

Utility Trench Backfill

Future contractors should strictly adhere to specifications set forth in the State of California Construction Safety Orders for "Excavations, Trenches, Earthwork". For the purpose of this section of the report, bedding is defined as material placed in a trench up to one foot above a utility pipe, and backfill is defined as all material placed in a trench above the bedding.

Unless concrete bedding is required around utility pipes, free-draining sand should be used as bedding. Sand proposed for use in bedding should be tested in our laboratory to verify its suitability and to measure its compaction characteristics. Sand bedding should be compacted by mechanical means to achieve at least 70 percent relative density based on ASTM Tests D4253 and D4254.

Approved, onsite, inorganic soil or imported materials may be used above the base as utility trench backfill. If imported material is proposed for this use, a sample should be tested and approved by the project geotechnical consultants before any is delivered to the site.

Proper compaction of trench backfill will be necessary under and adjacent to certified compacted fill, building foundations, concrete slabs, and vehicle pavements. In these areas, backfill should be conditioned with water to produce a soil-water content of about two percent above optimum content and placed in horizontal layers not exceeding six inches in thickness (before compaction).

Each layer should be compacted to at least 90 percent relative compaction based on ASTM Test D1557. The upper 12 inches of trench backfill under vehicle pavements should be compacted to at least 95 percent relative compaction.

Where any trench crosses the perimeter foundation line of any building, the trench should be completely plugged and sealed with compacted clay soil for a horizontal distance of two feet on either side of the foundation.

FOUNDATIONS

Conventional Spread Footings: The foundation of the proposed structure may be comprised of conventional spread (continuous and pad) footings, founded a minimum of 24 inches into the dense natural soil exposed in the subterranean garage excavation. All continuous footings shall be reinforced with a minimum of two #4 steel bars placed near the top and bottom of the footings. Reinforcement for pad footings shall be specified by the project civil/structural engineer. Footings may be sized utilizing the following design parameters:

FOUNDATIONS BEARING INTO DENSE NATURAL SOIL

Foundation Type	Minimum Width (Inches)	Maximum Vertical Bearing (PSF)	Allowable Coefficient of Friction	Allowable Passive Earth Pressure (PSF)	Maximum Lateral Earth Pressure (PSF)	Minimum Embedment Depth (Inches)
Continuous	18	2000	0.30	300	4000	24
Pad	24	2500	0.30	300	4000	24

The bearing values may be increased by 20 percent for each additional foot of width or depth to a maximum bearing capacity of 3000 psf for continuous footings, and 4000 psf for pad footings.

The bearing values presented above are net bearing values; the weight of concrete below grade may be neglected. Embedment depths should be measured from the lowest adjacent grade.

During foundation construction, care should be taken to minimize evaporation of water from foundation and floor subgrades. Scheduling the construction sequence to minimize the time intervals between foundation excavation and concrete placement is important. Concrete should be placed only on foundation excavations that have been kept moist and free from drying cracks and that contain no loose debris or soil.

LATERAL DESIGN

The bearing values provided above include the total dead plus frequently applied live loads. This value may be increased by a factor of one-third for short duration loading, such as the effects of wind and seismic forces. When combining passive pressure and friction for lateral resistance, the passive component should be reduced by a factor of one-third.

FOUNDATION SETTLEMENT

Settlement occurs as a result of stresses imposed on a soil. Typically, the most significant stress is the weight of structure(s). However, significant introduction of moisture into the subgrade may also induce settlement. Water can infiltrate the soil pore space, increasing the weight and softening the soil. In a granular soil, water introduction may result in collapse (hydroconsolidation), causing settlement.

Many factors are involved in the susceptibility of a soil to settlement. In the case of a clayey soil, the initial dry density and moisture content essentially govern the engineering behavior. If the clayey soil possesses a very low density, the soil could collapse, causing settlement upon saturation. However, given the same soil at a very high density, the soil will likely possess expansive characteristics. Accordingly, depending on the initial density and moisture content, the clayey soil could either settle or expand with moisture introduction.

In the case of fine-grained soils, generally, settlement is not an instantaneous process, but is time dependent. Permeability, as well, is an important factor in the behavior of fine-grained soils. Under saturated conditions, with an application of a static load, such as weight of the structure(s), the water will carry the load with the development of a pressure gradient that will drive out the pore water, and correspondingly, consolidation results with the dissipation of the pore water. The time required for water to drain from a soil layer is a function of the permeability. The higher the permeability, the faster the pore pressure will dissipate and the faster the soil will settle. In the case of loose, dry granular soil, settlement occurs as moisture is introduced. The rate of settlement is largely dependent on the rate of moisture introduction.

Based on the anticipated foundation loading and corresponding foundation design, in accordance with the preceding sections of this report, maximum and differential settlement are not expected to exceed 3/4 inch and 3/8 inch (in 20 feet), respectively. The majority of the settlement should occur during the construction phase, with post construction settlement being within acceptable ranges for the proposed type of structure.

RETAINING WALLS

Standard cantilevered retaining walls and braced walls below grade may be designed utilizing the following parameters. The design parameters presented below incorporate the results of our evaluation of active soil pressures utilizing anticipated wall loadings and height. Calculations are included in Appendix E.

A. The average bulk density of material placed on the backfill side of the wall will be approximately 125 pcf.

B. Equivalent fluid pressure values utilized in design of the cantilevered retaining walls should be applied as follows:
   - 48 pcf/ft for 2:1 slope behind the retaining wall
   - 35 pcf/ft for level backfill behind the retaining wall

C. Braced walls without surcharge and with a level backfill should be designed to resist an active earth pressure of 55 pcf.

D. These values are for non-seismic conditions and are based on the assumption that the wall backfill will be adequately drained. An increase in these pressures may be necessary if vehicular traffic or any building structures are to be located adjacent to the retaining wall. Construction traffic and compaction equipment should be kept a minimum of three feet from the retaining wall unless these surcharges are accounted for in the design.

E. A zone of drainage material at least 12 inches wide should be placed on the backfill side of the wall and should extend upward to within 24 inches of the top of the backfill. Adequate drainage devices should be utilized to collect water that may accumulate in the backfill and transport and discharge at the appropriate locations. The upper 18 to 24 inches of backfill should consist of a compacted clayey soil. We recommend that surface drainage, such as a concrete-lined V-ditch, be provided immediately behind the retaining wall that diverts water away from the area and discharges at a suitable location.

F. Wall backfill areas not occupied by specified drainage materials should be backfilled with certified compacted fill placed as specified above under "Site Preparation, Grading and Compaction of the Utility Trench Backfill".

G. Subdrains shall consist of Schedule 40, perforated SDR-35 PVC pipe placed with the perforations downward in a blanket of 3/4" durable aggregate such that the subdrain pipe is surrounded by a minimum six inches of gravel on all sides. The gravel blanket shall be wrapped with a geosynthetic filter fabric such as Mirafi 140 or a suitable equivalent. Fabric joints should be overlapped a minimum of three feet. Minimum specifications for pipe diameter, aggregate volume and fabric width are provided as follows:

Run Length (ft)	**Pipe Diameter (in)**	**Aggregate Volume (ft^3)**	**Fabric Width (ft)**
0 - 200'	4"	1.7	10.5'

The project geotechnical consultant should observe and approve all subdrain installations prior to placing compacted fill.

CONCRETE SLABS-ON-GRADE

New reinforced concrete slabs-on-grade should be a minimum of five-inches thick and should be reinforced with a minimum of #4 bars spaced at 16 inches on center in each direction. Concrete shall be cast over a minimum four-inch thickness of sand, placed over a minimum eight inches of certified compacted subgrade in the subterranean garage excavation prior to placing the reinforced concrete.

A modulus of subgrade reaction and "K" of 200 pounds per cubic inch may be used for design of the slab-on-grade, provided the subgrade soils below the garage slab are compacted to a minimum eight inches below the finish subgrade. Areas of moisture-sensitive floor covering should be provided with a 10- mil visqueen moisture barrier. The vapor barrier should be placed near the center of the sand layer, a minimum of two inches below the concrete slab.

Non-supported edges should be provided with a thickened slab edge, which has nominal dimensions of eight inches in width and 12 inches in depth. The thickened slab edge should be reinforced with a minimum of one #4 bar placed near the top and bottom of the thickened slab edge.

Recommendations presented in the American Concrete Institute should be complied with for all concrete placement and curing operations. Improper curing techniques and/or excessive slump (water-cement ratio) could cause excessive shrinkage, cracking or curling. Concrete slabs should be allowed to cure adequately before placing vinyl or other moisture-sensitive floor coverings.

CHEMICAL TESTING

Selected samples were collected and tested by Columbia Analytical Services, Inc., for sulfate, chloride, resistivity, etc. Concrete mix design, including water cement ratio, should be in accordance with the 2001 CBC Table 19-A-04. The chemical test results are included herewith in Appendix C.

EXPANSIVE SOILS

Expansion tests performed in accordance with ASTM Standard 4829 "Expansion Index Test", indicate the onsite soil has a low-expansive soil condition, E.I.=0-26. Accordingly, foundations for the proposed improvements should be designed for low-expansive soil conditions.

TEMPORARY EXCAVATIONS AND SHORING

For preliminary planning purposes, all excavations that exceed five feet in vertical height should be supported by a temporary shoring system. Various methods of shoring may be used, including internally braced or externally tied back shoring. The use of externally tied-back shoring would result in an unobstructed construction area. Braced or tied-back shoring should be designed to resist a trapezoidal earth pressure distribution as shown on the loading diagram in Appendix E.

Soldier piles may be utilized to support temporary excavations over five feet in height. Soldier piles should be a minimum 24 inches in diameter and shall be placed no greater than eight feet center to center, and should be designed in accordance with the allowable lateral bearing values (passive value) provided in the preceding sections. Cantilevered soldier piles shall be designed for an active fluid pressure of 30 pounds per cubic foot per lineal foot of tributary area.

The shoring may be tied back with drilled friction anchors. For design purposes, it may be assumed that the potential active plane of failure is determined by a plane drawn at 35 degrees with the vertical at the bottom of the excavation. Friction anchors may be designed utilizing an allowable skin friction of 500 pounds per square foot.

The anchors should be installed at angles of 15 to 40 degrees below the horizontal. At least four of the initial anchors should be selected and tested to 200 percent of the design load to verify the design capacity. All the anchors should be pre-tested to at least 150 percent of the design load. The deflection under the test load (150 percent of the design load) should not exceed 0.1 inch over a 15 minute period in order for the anchor to be approved for the design capacity; the total deflection during the test load should not exceed 12 inches.

Structural concrete should be placed in the lower portion of the drilled shaft up to the assumed failure plane. The anchor shaft between the failure plane and the face of the shoring should not be backfilled with concrete until after the anchor has been tested and the design load placed on the anchor. If desired, this portion of the shaft could be backfilled with sand prior to testing the anchor. Concreting of the anchor should be done by pumping the concrete into the bottom of the shaft.

In addition to the recommended earth pressure, the shoring adjacent to the street should be designed to resist a uniform lateral earth pressure of 100 pounds per square foot, acting as a result of an assumed 300-pound per foot surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 12 feet from the shoring, the traffic surcharge may be neglected.

Shoring should also be designed to resist surcharge loading imposed by adjacent structures. A coefficient of 0.4 may be used to convert the surcharge load in a uniformly distributed force to the shoring if the adjacent structures are within a distance equal or less than the height of shoring. Lagging should be applied between shoring piles and should be designed for the earth pressure not to exceed 400 pounds per square foot.

The installation of temporary shoring and testing of the tie-back anchors should be observed and approved by a representative of this office. It is recommended that the shoring plans and specifications be reviewed by this office. Each prospective shoring contractor may suggest modifications to the above recommendations. We will be pleased to work with the selected shoring contractor to establish design data for alternate procedures.

The geotechnical consultant should be present during grading to observe the temporary excavation. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations, nor to flow towards it. No vehicular surcharge shall be allowed within five feet of the top of the cut.

DRAINAGE AND MOISTURE PROTECTION

The site should be fine graded to direct drainage away from any structures. Drainage should not be allowed to pond anywhere on the pad, against foundations or pavements, and should be directed toward suitable collection discharge facilities.

To promote the rapid drainage of surface water from pavements and to minimize the risk of water ponding on pavements, we recommend that pavements be designed with surface gradients of at least one percent along principal directions of drainage. Water seepage or the spread of extensive root systems into the soil subgrades of foundations, slabs or pavements could cause differential movements and consequent distress in these structural elements. This potential risk should be given consideration in the landscape design.

Walls located below grade have a history of moisture intrusion and leakage. Conventional water proofing materials, such as asphalt emulsion have often proved ineffective. Certain precautions can be taken to reduce the possibility of future water proofing problems. Super plasticized and water retardant concrete may be utilized to make pouring easier and reduce cracking and shrinkage. Water proofing paints, such as "Thoroseal" may be used, as they have been proven more effective than conventional asphalt emulsion. It is recommended that the project architect provide water-proofing specifications for all below grade walls and structures.

ADDITIONAL SERVICES

It is recommended that this office be provided an opportunity for a general review of the final design plans and supporting documents for overall compliance with the recommendations presented in this report. Additionally, this office should be retained to provide services during grading, foundation excavation and overall construction phases of the project. Observation of foundation excavations should be performed prior to the placement of concrete and reinforcing steel to confirm that the foundations are founded in the recommended bearing materials. Field and laboratory testing of compacted fill should be performed to verify compliance with recommendations presented herein.

PLAN REVIEW

CalWest Geotechnical should review all final design plans and supporting documents. This will allow us to perform a general review for compliance with recommendations presented in this report.

SITE OBSERVATIONS

Prior to the start of construction, we recommend that a pre-construction meeting be held with the contractor to discuss the project and that a representative of CalWest Geotechnical be present at that meeting. We further recommend that CalWest Geotechnical should perform the following tasks prior to, and/or during, construction of the project:

1. Review all final design plans and supporting documents;
2. Observe and advise during clearing and stripping of the site;
3. Observe and advise during excavations of the existing soil;
4. Observe, test and advise during all excavations, installation of subdrainage systems, all grading, and placement of certified compacted fill;
5. Observe the construction of all temporary excavations and temporary shoring systems;
6. Observe foundation excavations and slab subgrades;
7. Observe installation of retaining wall subdrains and backfill;
8. Observe and test during placement of utility trench backfill.

ACKNOWLEDGEMENTS

California, historically, has experienced major destruction due to storms, flooding, firestorms, and earthquakes. The design of drainage control devices is based on rainfall records and the requirements of the authoritative building department agencies. Even so, the capacity of drainage devices often is exceeded, which results in considerable damage. Slopes associated with hillside developments, which have performed satisfactorily over a long period of time, in a majority of cases, could fail as a result, even though such slopes have been designed to the minimum standards set forth by the California Building Code or other authoritative codes.

As for the design of earthquake forces, the records on which engineering design is based have been accumulated over a relatively short time frame. Every earthquake provides new information and data as to the cause and effect of large earthquakes. As an example, the January 17, 1994 Northridge earthquake recorded ground accelerations that exceeded all previous earthquake records. In addition, the engineering industry has learned that there are many blind-thrust faults present in Southern California. The presence of these faults were known by petroleum geologists, but without much significance attached to the information by seismologists.

It should be understood that residential and commercial structures are constructed to the minimum standards as set forth by the California Building Code and other authoritative codes. Higher standards are utilized for hospitals, schools, and other critical structures, that must remain serviceable in the event of a disaster. Generally, Building Code requirements provide minimum standards to prevent catastrophic failure. Accordingly, it is believed that site structures are not likely to collapse, although considerable damage may occur.

PROPERTY OWNER'S RESPONSIBILITY

The property owner should care for drainage around the site structures and all graded slopes. To maintain the continued effectiveness of onsite drainage devices, there are important procedures that must be undertaken by the property owner on a regular basis. These procedures are specifically for drainage and debris protection, and therefore, the procedures should be performed prior to each rainy season, with sufficient time to allow for thorough maintenance.

In addition to maintenance of drainage devices, an inspection should be made for rodent activity. Small, burrowing rodents, such as ground squirrels and gophers, create avenues for infiltration of surface water, which could create surficial slope failures. Evidence of rodent infestation should result in the employment of a licensed exterminator. It should be emphasized that these procedures may require periodic performance if reinfestation occurs.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is prepared for use by DIDM Development Corporation and their authorized agents and should not be considered transferable. Prior to use by others, the subject site and this report should be reviewed by CalWest Geotechnical to determine if any additional work is required to update this report.

The findings presented in this report are valid as of this date and may be invalidated wholly or partially by changes outside our control. Therefore, this report should be subject to review and should not be relied upon after a period of one year or if any significant changes are made.

It is the intent of this report to aid in the design and construction of the described project. Implementation of the advice presented in the "Conclusions and Recommendations" sections of this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual conditions will not be discovered during or after construction.

The conclusions and recommendations contained within this report are based on field observations of the site conditions. Recommendations are based on the assumption that the subsurface conditions do not deviate appreciably from those indicated by the individual test pits placed on the subject property. If conditions encountered during construction appear to differ from those described in this report, this office should be notified so we may determine if any modifications are necessary. In this way, any required supplemental recommendations can be made with a minimum delay to the project.

The recommendations are based on preliminary information provided to us at the start of the investigation. Any changes of this information may require additional work. This report has been prepared in accordance with generally accepted engineering practices and makes no warranties, either express or implied, as to the professional advice provided under the terms of the agreement and included in this report.

Respectfully submitted,

REGISTERED PROFESSIONAL ENGINEER
LEONARD T. LISTON
No. 31902
Exp. Dec. 2008
CIVIL
STATE OF CALIFORNIA

Leonard Liston
President
RCE 31902

Eli Katibah
Staff Engineer

Enc: Appendix A - Site Location Map
Appendix B - Logs of Exploratory Test Pits
Appendix C - Laboratory Test Procedures/Results
Appendix D - Geotechnical Map and Cross-sections
Appendix E - Foundation Bearing Capacity Analyses- Pressure Distribution For Brace Loads

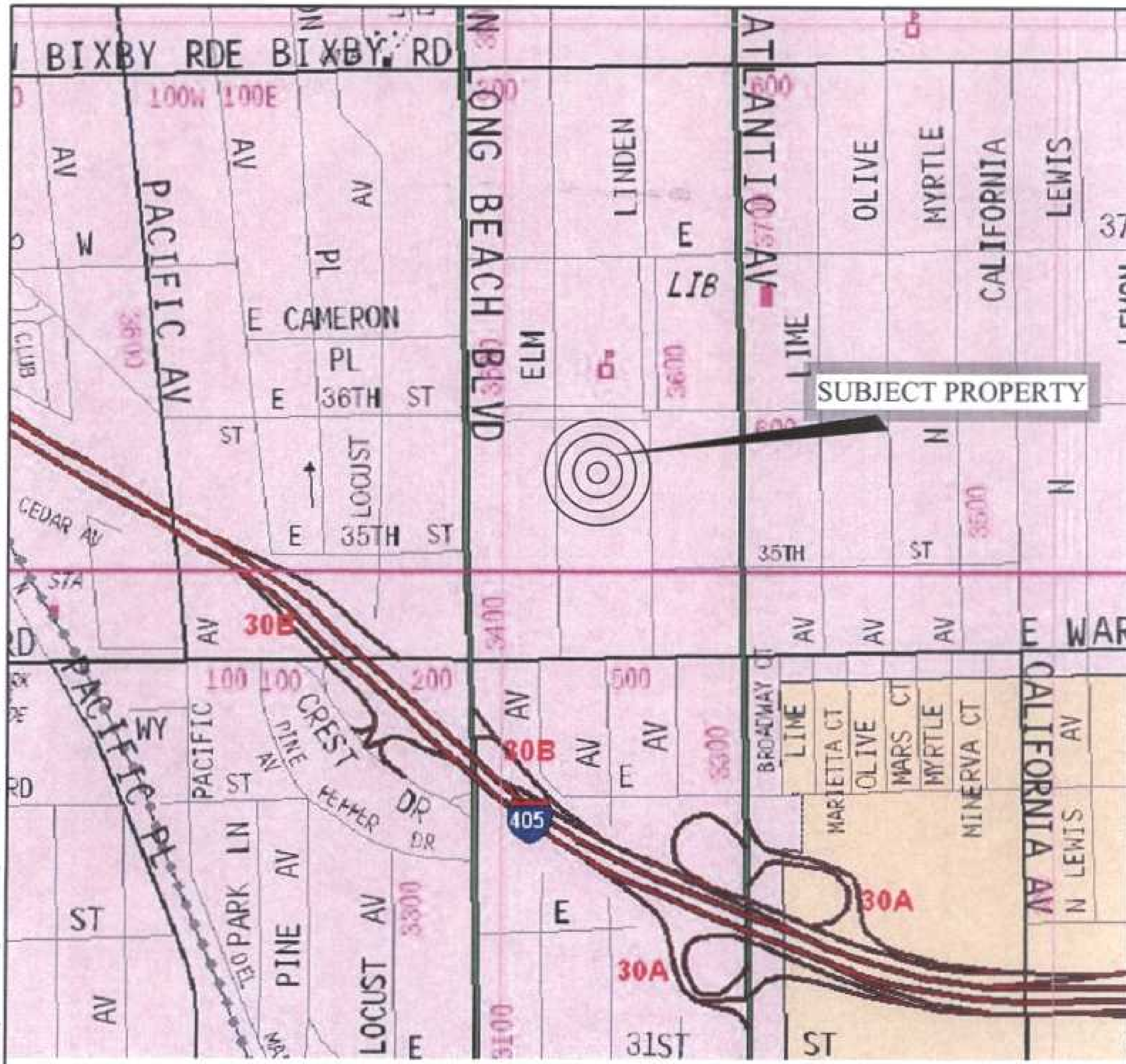
APPENDIX A

CALWEST GEOTECHNICAL
CONSULTING ENGINEERS
A DIVISION OF LC ENGINEERING, INC.
889 PIERCE COURT, SUITE 101
THOUSAND OAKS, CA. 91360
(818)991-7148
(805)497-1244

PROJECT: D.I.D.M. Development Corp
ADDRESS: 3635 Elm Ave, Long Beach
DRAWN: RH REF.: 765 E7
DATE: October, 2007 JOB #: G 4972

VICINITY MAP

SHEET TITLE



N
NORTH

REFERENCE: THOMAS BROTHERS MAP GIDE, PAGE: 765

CALWEST GEOTECHNICAL
CONSULTING ENGINEERS

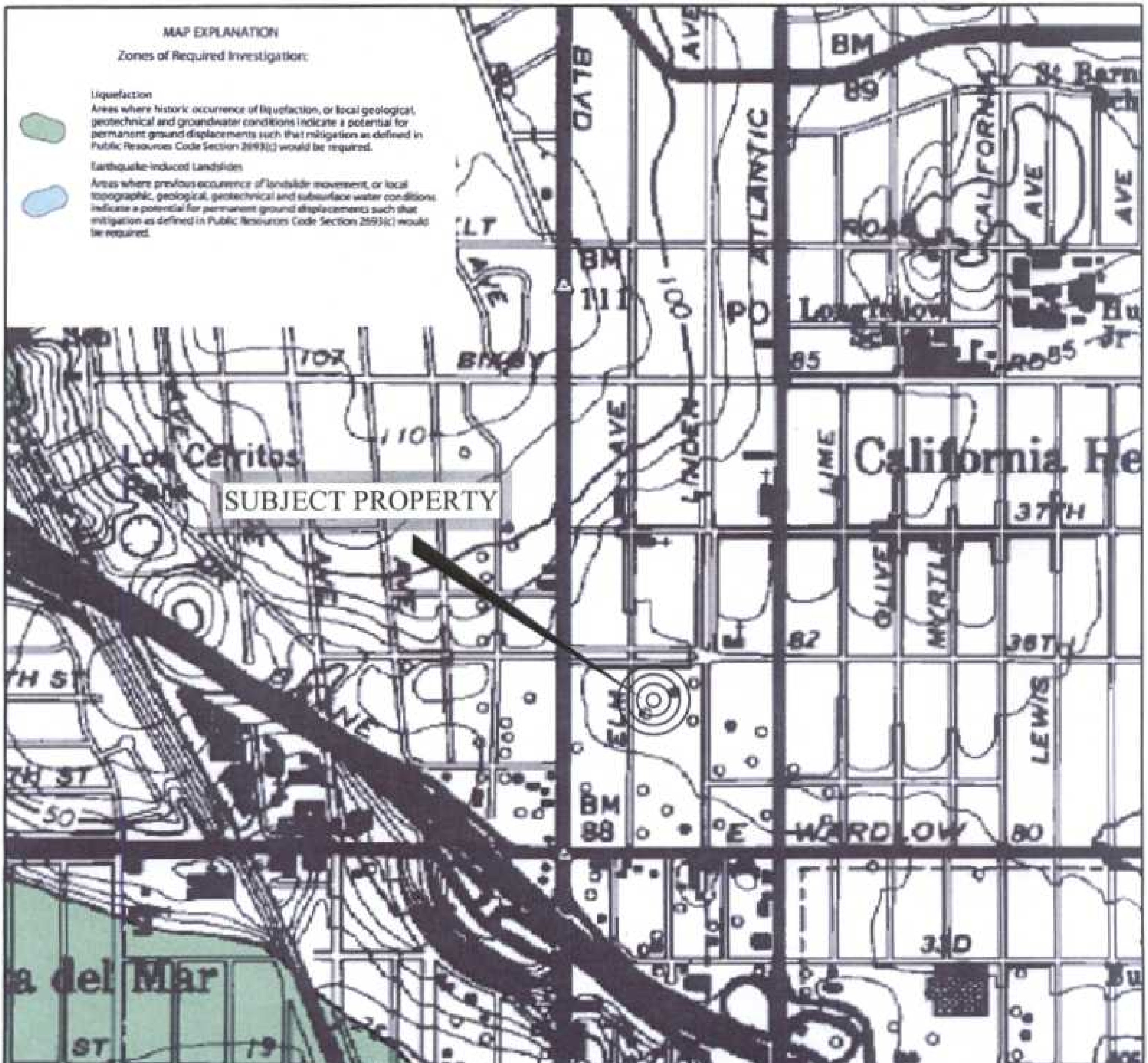
A DIVISION OF LC ENGINEERING, INC.

889 PIERCE COURT, SUITE 101
THOUSAND OAKS, CA 91360
(818)991-7148
(805)497-1244

PROJECT: D.I.D.M. Corporation JOB #: G 4972
ADDRESS: 3635 Elm Ave, Long Beach
DRAWN: RH DATE: CTOBER, 2007

SIESMIC HAZARD MAP

SHEET TITLE



N
NORTH

REFERENCE: SIESMIC HAZARD ZONE, LONG BEACH QUADRANGLE

APPENDIX B

EXCAVATION DATA

PROJECT DIDM **JOB NO:** G 4972

EXCAVATION METHOD 6" Dia. HALLOW STEM AUGER **DATE:** Sept., 07

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET		DESCRIPTION: B-1
						1	--	
						2	--	0-5' **SOIL:** Silty Sand, orange brown, slightly moist,
						3	--	fine grained, dense.
						4	--	
5'	R	SM	10.3	109.4	22,44,40	5	--	
						6	--	
						7	--	5' - 10' **SOIL:** Silty Sand, yellowish orange brown,
						8	--	moist,fine grained, dense.
						9	--	
10'	R	SM	10.1	110.3	25,45,40	10	--	
						11	--	
						12	--	10' - 15' **SOIL:** Clayey Sand, grayish yellowish brown,
						13	--	slightly moist, dense.
						14	--	
15'	R	SC	12.9	112.4	35,50/5"	15	--	
						16	--	
						17	--	15' - 20' **SOIL:** Clayey Sand, grayish brown,
						18	--	slightly moist, dense.
						19	--	
20'	R	SC	11.2	111.7	33, 50/5"	20	--	
						21	--	
						22	--	
						23	--	20' - 30' **SOIl:** Clayey Sand, grayish brown, moist,dense,
						24	--	occasional subangular pebbles to gravel size rocks.
25'	R	SC	13.4	108.9	34,34,48	25	--	
						26	--	
						27	--	
						28	--	
						29	--	
30'	R	SC	14.6	109.2	19,33,42	30	--	

REFERENCE: C:\OWINFILE\LAB\LOG.XLS

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THOUSAND OAKS, CA
(818) 991-7148
(805) 497-1244

EXCAVATION DATA

PROJECT DIDM **JOB NO:** G4972

EXCAVATION METHOD 6" dia. HALLOW STEM AUGER **DATE:** Sept., 07

DESCRIPTION: B-1 (CONTINUED)

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION
						31	
						32	
						33	30' - 35' **SOIL:** Silty Sand, dark gray, moist,
						34	fine grained, dense.
35'	R	SM	14.8	109.7	14,22,48	35	
						36	35'- 40' **SOIL:** Fine grained, Silty Sand, dark bluish gray,
						37	moist, dense. Minor water seep at 38'
						38	
						39	
40'	R	SM	17.3	107.8	12,30,44	40	
						41	
						42	
						43	
						44	
45'	R	SM	15.3	110.2	14,23,41	45	40' - 50' **SOIL:** Silty Sand, dark gray, slightly moist,
						46	dense.
						47	
						48	
						49	
50'	R	SM	13.8	111.3	38,50/5"	50	
							END @ 50'
							NO GROUNDWATER
							NO CAVING

REFERENCE: C:\OWINFILE\LAB\LOG.XLS

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EXCAVATION DATA

PROJECT DIDM **JOB NO:** G 4972

EXCAVATION METHOD 6" Dia. HALLOW STEM AUGER **DATE:** Sept., 07

DESCRIPTION: B-2

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION
						1	
						2	0-5' **SOIL:** Silty Sand, grayish brown, slightly moist,
						3	medium dense to dense, small roots.
						4	
5'	(SPT)				8,11,17	5	
						6	
						7	5' - 10' **SOIL:** Silty Sand, grayish brown to orange brown,
						8	moist, fine grained, dense.
						9	
10'	R	SM	14.3	111.4	24,33,47	10	
						11	
						12	10' - 15' **SOIL:** Clayey Sand, grayish brown,
						13	slightly moist, well cemented, dense.
						14	
15'	(SPT)				16,23,24	15	
						16	
						17	15' - 20' **SOIL:** Clayey Sand, light brown to grayish brown,
						18	slightly moist, dense.
						19	
20'	R	SC	10.4	112.6	33,50/5"	20	
						21	
						22	
						23	20' - 25' **SOIL:** Clayey Sand, grayish brown, moist,dense,
						24	occasional subangular pebbles to gravel size rocks.
25'	(SPT)				20,30,45	25	
						26	
						27	
						28	25' - 30' **SOIL:** Clayey Sand, grayish brown, moist,dense,
						29	
30'	R	SC	12.7	110.8	50/5"	30	

REFERENCE: C:\OWINFILE\LAB\LOG.XLS

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EXCAVATION DATA

PROJECT DIDM **JOB NO:** G4972

EXCAVATION METHOD 6" dia. HALLOW STEM AUGER **DATE:** Sept., 07

DESCRIPTION: B-2 (CONTINUED)

SAMPLE DEPTH (ft)	SAMPLE TYPE	CLASSIFICATION UNIFIED SOIL SYSTEM	MOISTURE CONTENT (%)	IN PLACE DRY DENSITY (pcf)	BLOW COUNTS PER 6"	DEPTH IN FEET	DESCRIPTION
						31	
						32	
						33	30' - 35' **SOIL:** Silty Sand, grayish brown to medium brown,
						34	moist, fine grained, dense.
35'	(SPT)				12,15,23	35	
						36	35'- 40' **SOIL:** Silty Sand, dark gray,
						37	slightly moist, dense.
						38	
						39	
40'	R	SM	14.2	107.9	9,30,40	40	
						41	
						42	
						43	
						44	
45'	(SPT)				12,19,23	45	40' - 50' **SOIL:** Silty Sand, dark gray, slightly moist,
						46	dense.
						47	
						48	
						49	
50'	R	SM	14.9	109.6	22,50/4"	50	
							END @ 50'
							NO GROUNDWATER
							NO CAVING

REFERENCE: C:\OWINFILE\LAB\LOG.XLS

APPENDIX C

EXPLORATION AND LABORATORY TESTING PROCEDURES

CAL WEST GEOTECHNICAL

Exploration

Field exploration is performed utilizing a variety of equipment, such as; a truck-mounted rotary drill rig, a truck-mounted bucket auger drill rig, a track-mounted backhoe, a rubber-tire backhoe and hand labor. The earth materials encountered are continuously logged by our field engineer and/or geologist and classified by visual examination in accordance with the Unified Soil Classification System.

The locations of test pits are determined by field measurements utilizing the plans furnished by the client. The location of the test pits should be considered accurate only to the degree implied by the method used.

Undisturbed samples of soils encountered are obtained at frequent intervals. Samples are obtained from hand samplers. The soil is retained in brass rings of 2.50 inches inside diameter and 1.00 inches in height. The central portion of the sample is retained in close-fitting, waterproof containers.

Classification

The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the enclosed Log of Test Pits and Laboratory Plates.

Moisture-Density

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples. The information is useful in providing a gross picture of the soil consistency between test pits and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the enclosed Laboratory Plates. The field density and moisture content are determined as a percentage of the dry unit weight and are shown on the Log of Test Pits.

Shear Tests

Shear tests are performed in the Soil Test Direct Shear Machine per ASTM standard D3080, which is of the strain control type. Each sample is sheared under axial loads varying from 900 to 4000 lbs/sq. ft. in order to determine the Coulomb shear strength parameters, cohesion and angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are attached as graphic summaries on the enclosed Laboratory Plates.

Expansion Tests

In order to test the expansiveness of soil, a soil sample is compacted into a mold at near 50 percent saturation. A vertical confining pressure of 1-lbf/in is applied to the specimen and the sample is inundated with water. The deformation of the sample is measured over a 24-hour period or the rate of deformation becomes less than .0002 in./hr. whichever comes first. Results are shown on the enclosed Laboratory Plates.

COMPACTION / EXPANSION DATA

PROJECT: DIDM

JOB NO.: G4972 DATE: Sept. '07

TEST PIT NUMBER	SAMPLE DEPTH	SOIL TYPE	MAXIMUM DENSITY (PCF)	OPTIMUM MOISTURE (%)	EXPANSION INDEX
B-1	0-5'	Silty Sand	123.0	11.5	26

REFERENCE: C:\0TEMPLTE\SOILS\LAB\COMPEXP.XLS

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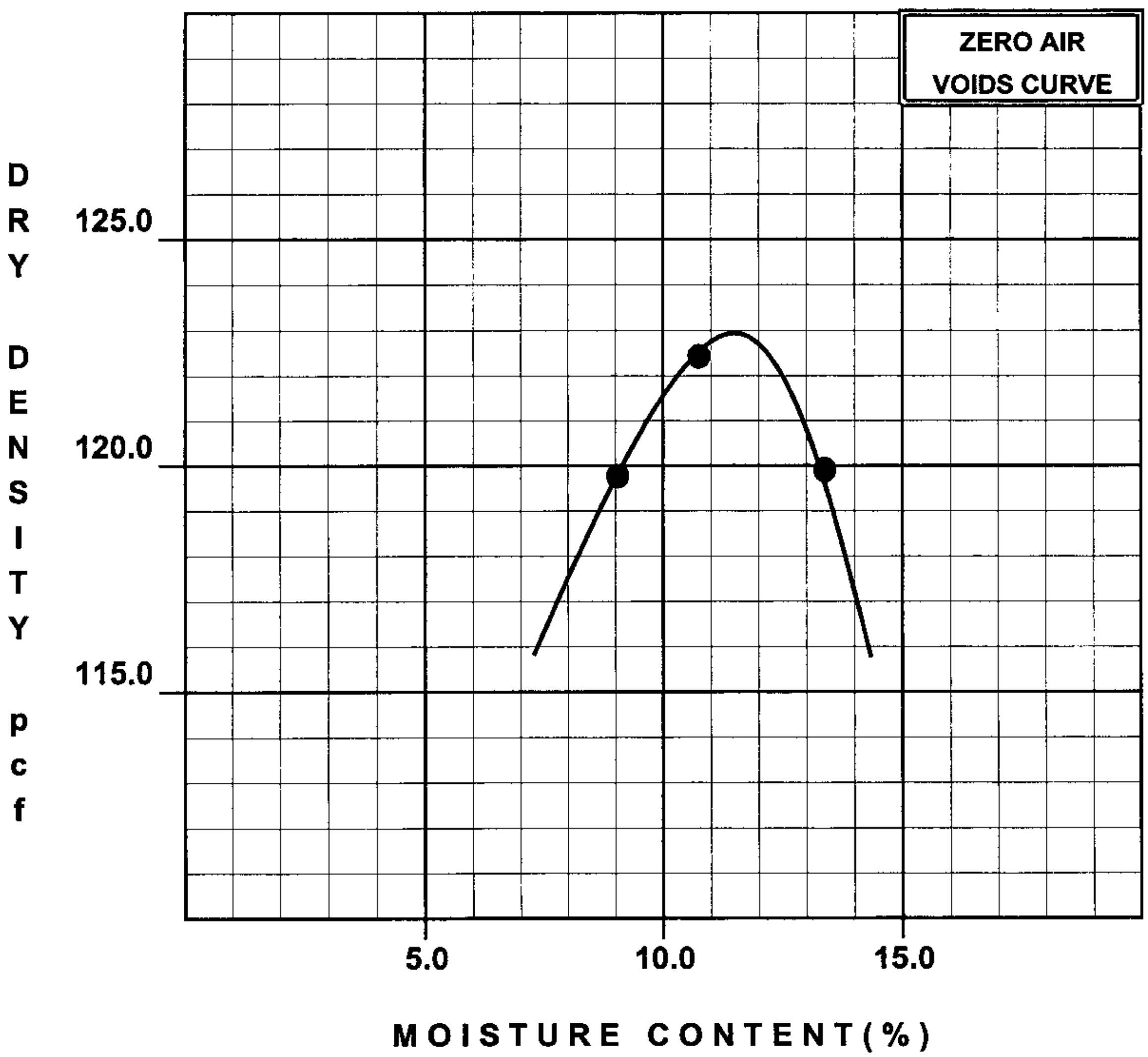
MAXIMUM DENSITY TEST

PROJECT NAME: DIDM **NUMBER:** G4972

SAMPLE (NO. & DEPTH)	TEST DESIGNATION	MAXIMUM DRY DENSITY	OPTIMUM MOISTURE CONTENT
BULK	ASTM D1557	123 (pcf)	11.5%



REFERENCE: C:\0TEMPLTE\SOILS\LAB\DENSITY.XLS

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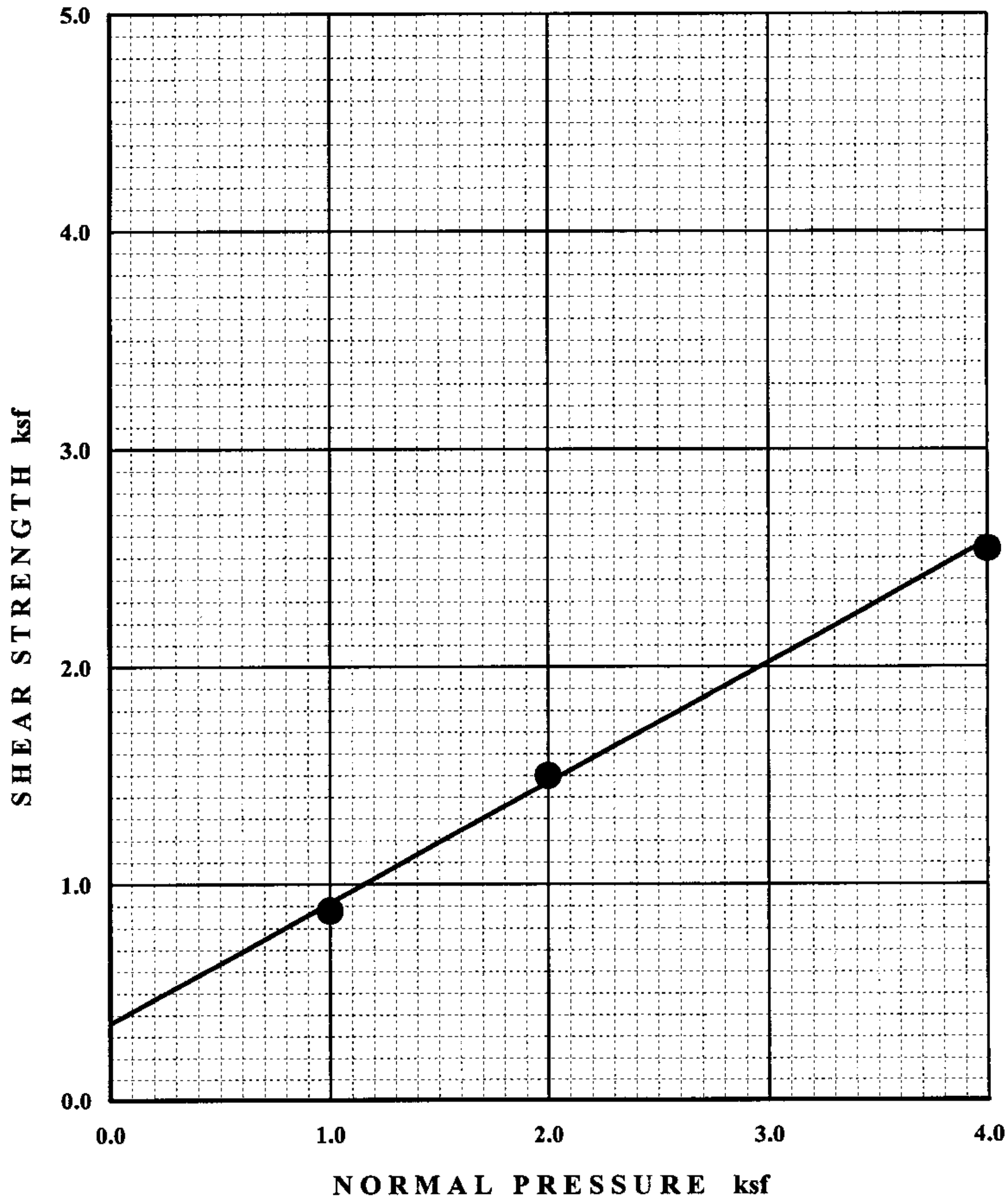
(805) 497-1244

SHEAR TEST DIAGRAM

PROJECT: DIDM **NUMBER:** G4972

SAMPLE: B-1 @ 10' (Silty Sand) **DATE:** Oct., 07



STRAIN RATE -0.005 in/min Initial moisture content = 10.1 %

SAMPLE SATURATION - 24 hrs Final moisture content = 17.7 %

DRY DENSITY & WATER CONTENT - 110.3 pcf @ 18 %

ULTIMATE SHEAR RESISTANCE

COHESION = 360 psf

PHI = 29 °

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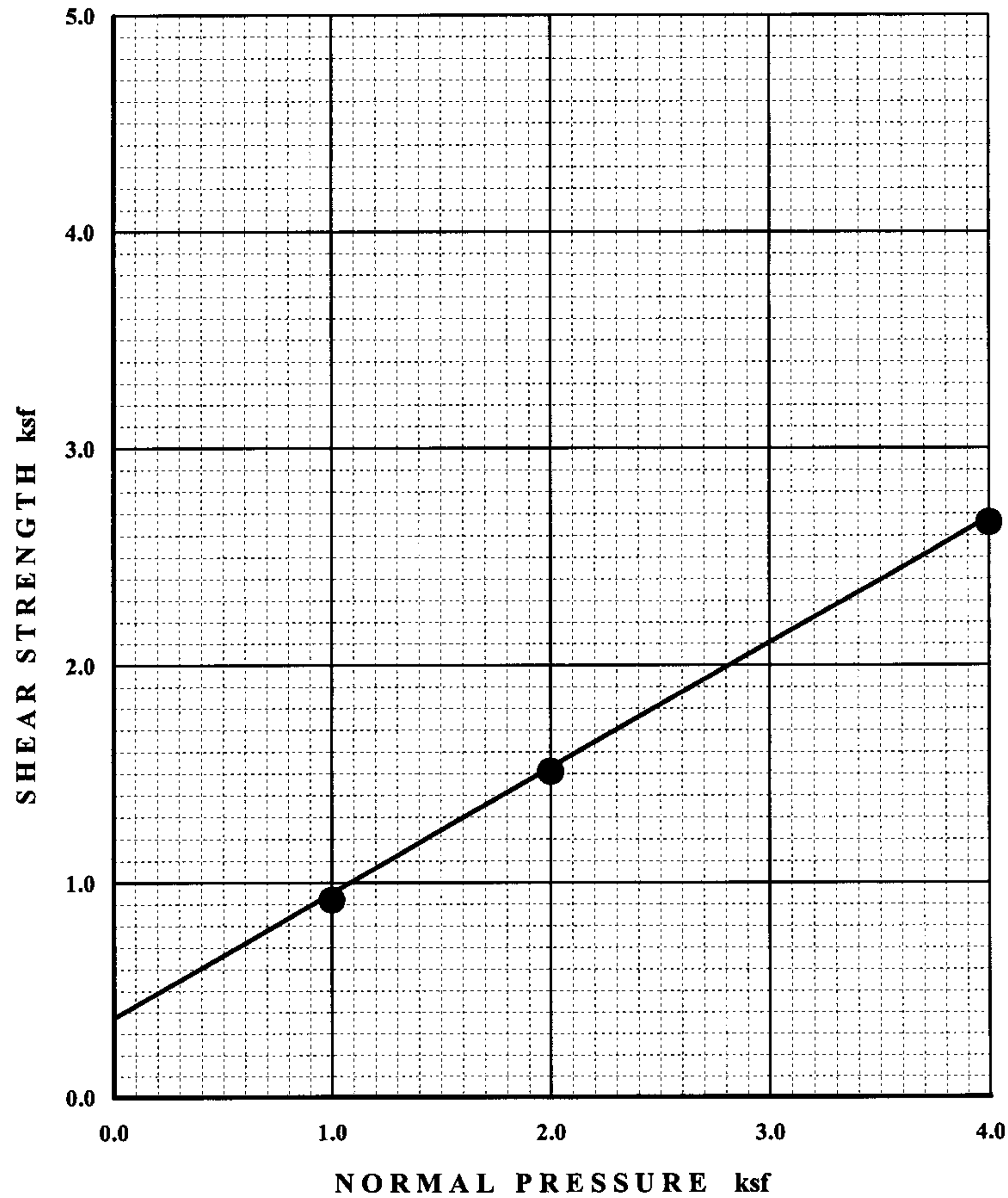
SHEAR TEST DIAGRAM

PROJECT: DIDM **NUMBER:** G4972

SAMPLE: B-2 @ 20' (Clayey Sand) **DATE:** Oct., 07



STRAIN RATE -0.005 in/min Initial moisture content = 10.4 %

SAMPLE SATURATION - 24 hrs Final moisture content = 16.6 %

DRY DENSITY & WATER CONTENT - 112.6 pcf @ 17 %

ULTIMATE SHEAR RESISTANCE

COHESION = 375 psf

PHI = 30 °

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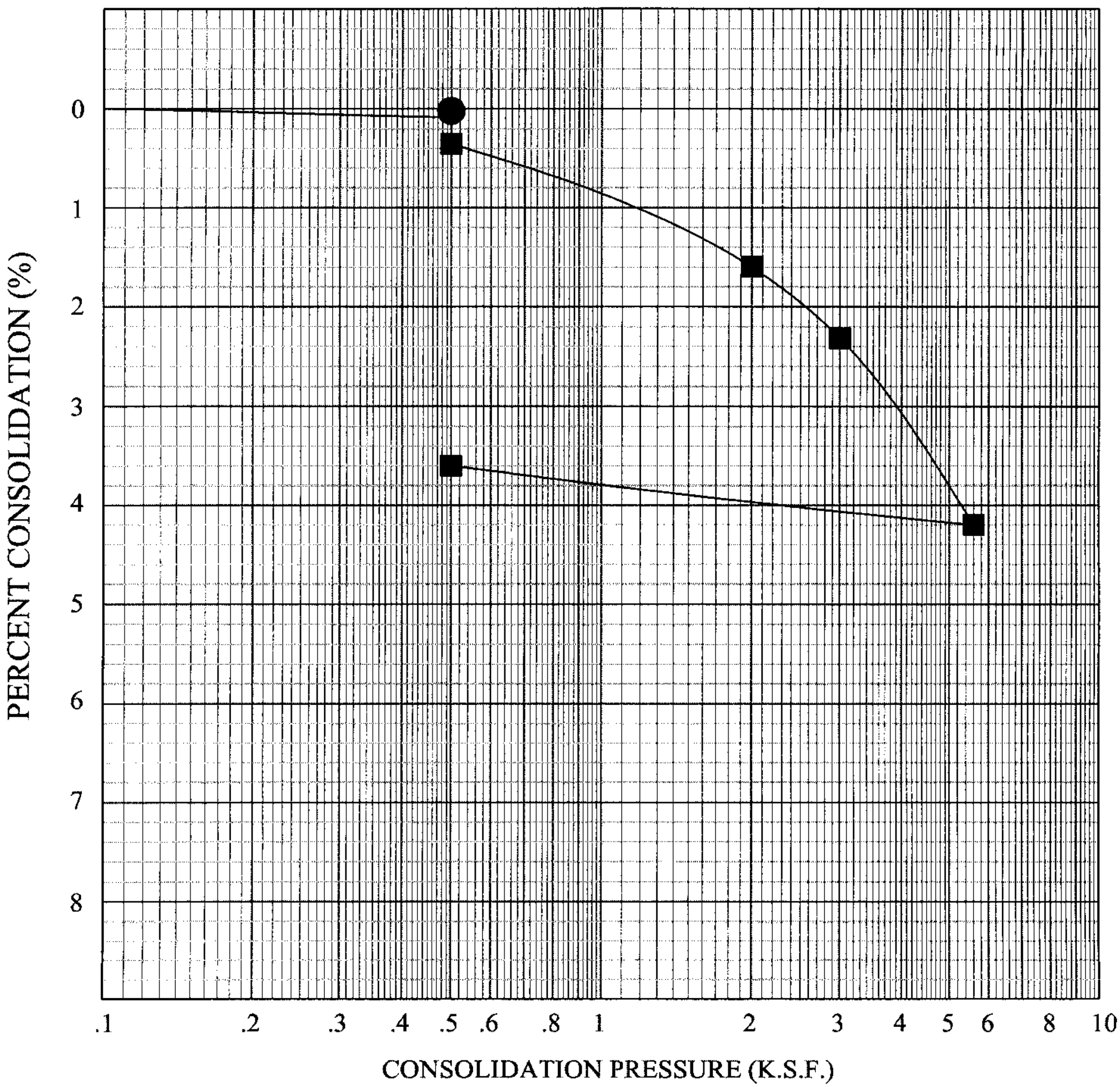
805-497-1244

PROJECT DIDM JOB NO. G 4972

SAMPLE B-1 @ 25 FT. DATE Oct., 2007

CONSOLIDATION TEST DIAGRAM



● = Insitu Moisture Content

■ = Saturated Condition

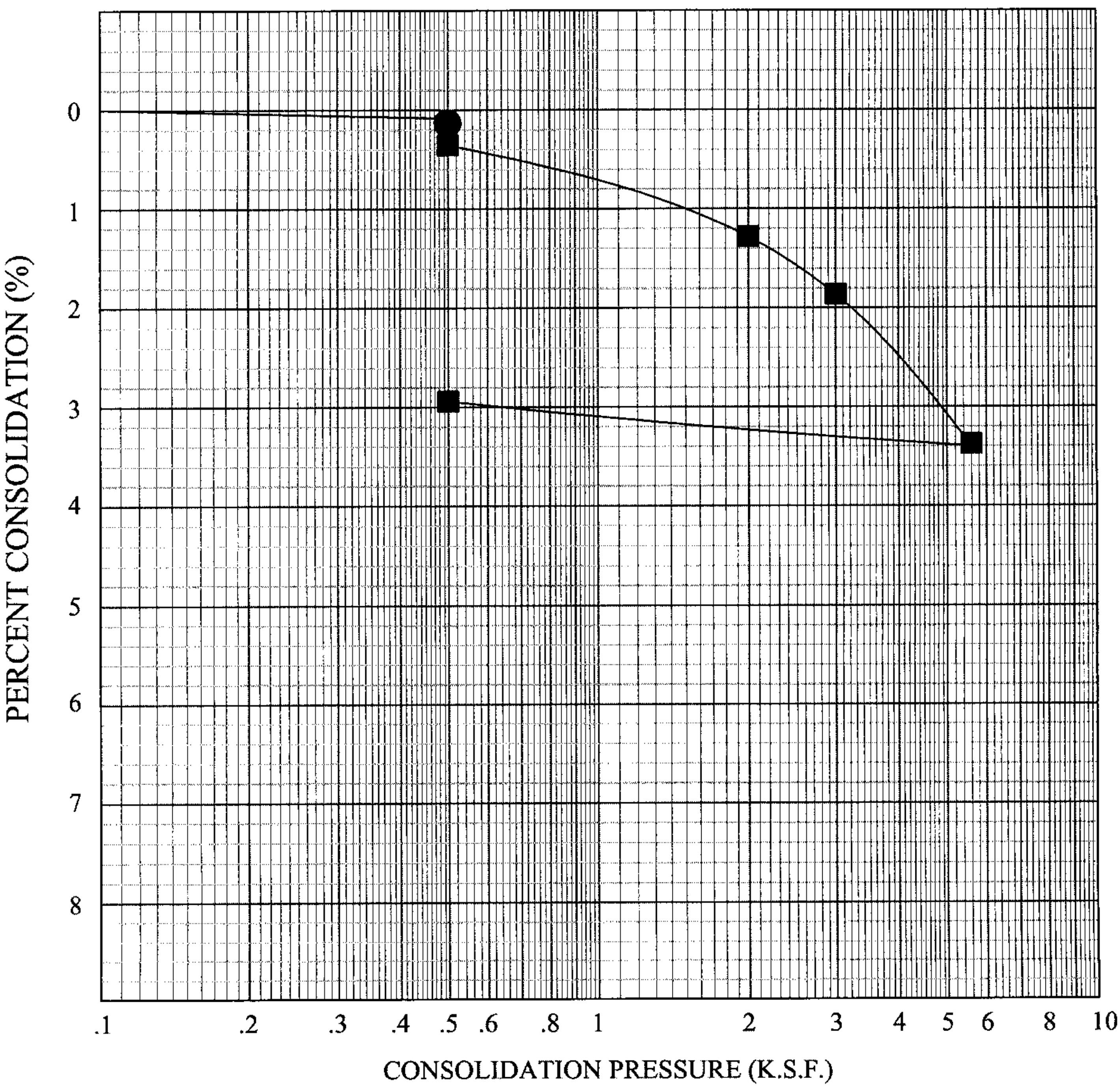
Initial Moisture Content 13.4%

Final Moisture Content 17.8%

PROJECT DIDM JOB NO. G 4972

SAMPLE B-1 @ 40 FT. DATE Oct., 2007

CONSOLIDATION TEST DIAGRAM



● = Insitu Moisture Content

■ = Saturated Condition

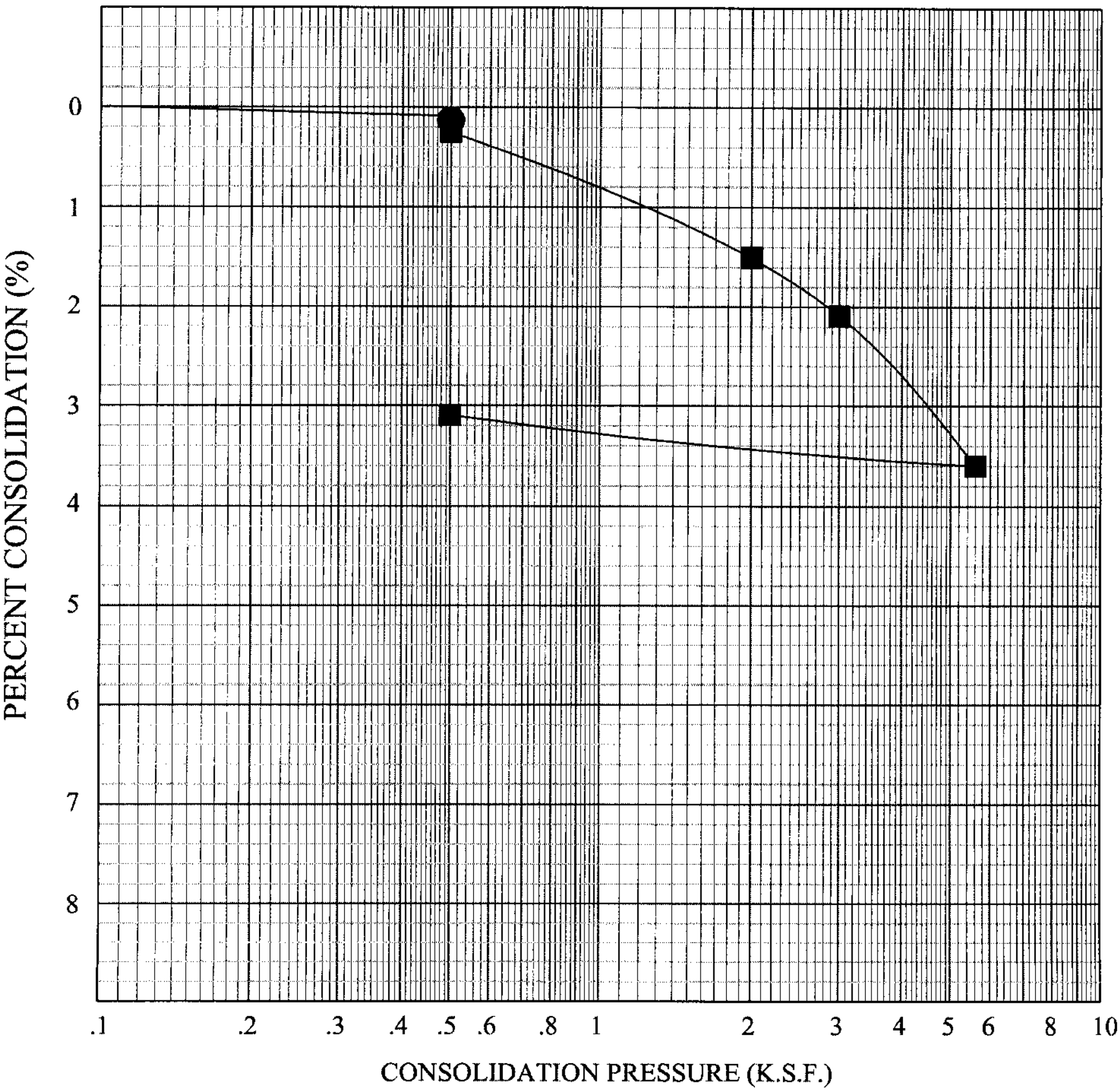
Initial Moisture Content 17.3%

Final Moisture Content 20.2%

PROJECT DIDM JOB NO. G 4972

SAMPLE B-2 @ 20 FT. DATE Oct., 2007

CONSOLIDATION TEST DIAGRAM



● = Insitu Moisture Content

■ = Saturated Condition

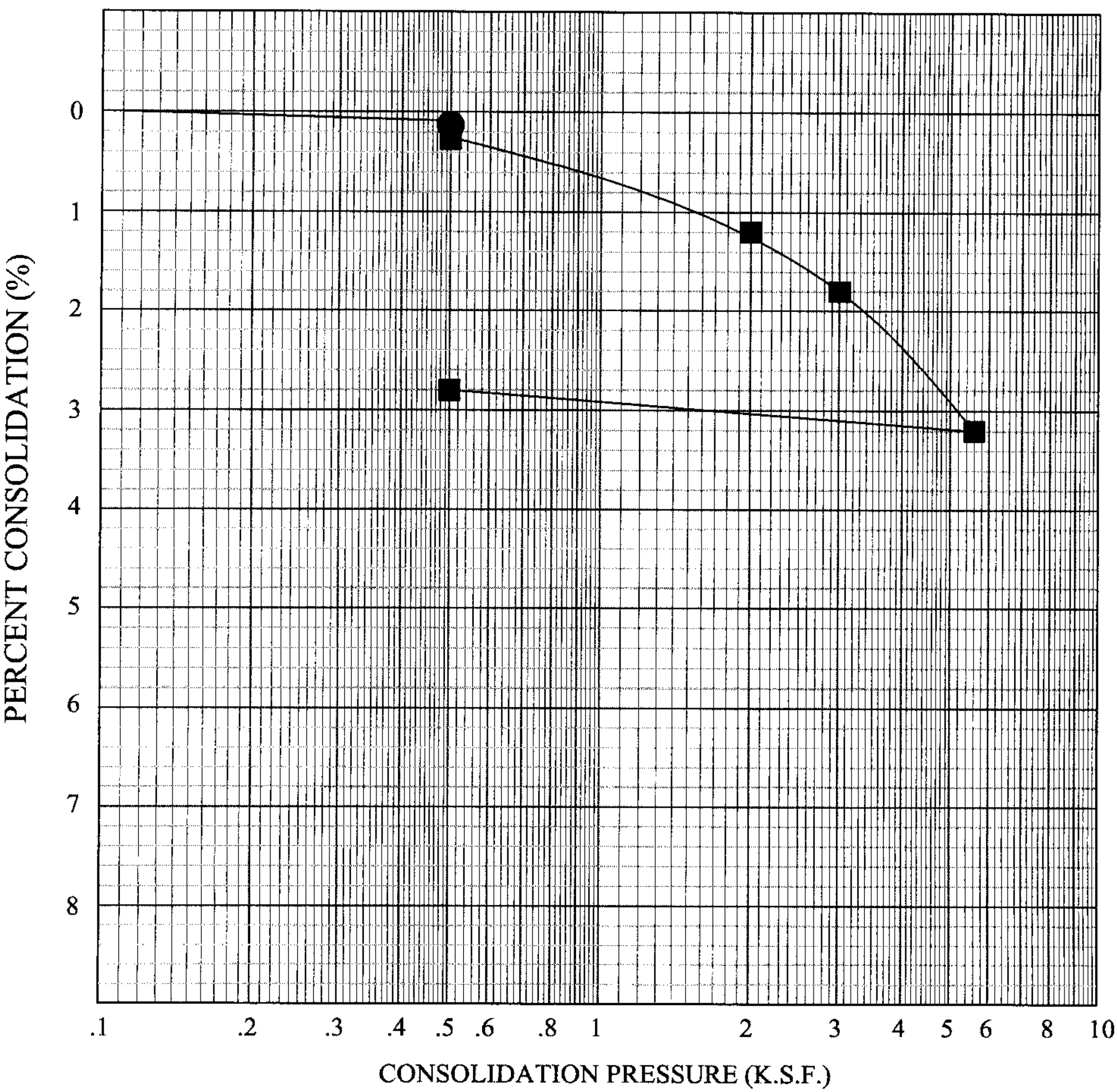
Initial Moisture Content 10.4%

Final Moisture Content 17.2%

PROJECT DIDM JOB NO. G 4972

SAMPLE B-2 @ 30 FT. DATE Oct., 2007

CONSOLIDATION TEST DIAGRAM



● = Insitu Moisture Content

■ = Saturated Condition

Initial Moisture Content 12.7%

Final Moisture Content 18.1%

TABLE 19-A-4 REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

SULFATE EXPOSURE	WATER-SOLUBLE SULFATE (SO_4) IN WATER, ppm	SULFATE (SO_4) IN WATER, ppm	CEMENT TYPE	Maximum Water-Cementitous Materials Ratio, by Weight, Normal-Weight Aggregate Concrete[1]	Normal Weight and Lightweight Aggregate Concrete, psi[1] x 0.00689 for MPa
Negligible	0.00 - 0.10	0 - 150	-	-	-
Moderate[2]	0.10 - 0.20	150 - 1,500	II, IP(MS), IS(MS)	0.5	4,000
Severe	0.20 - 2.00	1,500 - 10,000	V	0.45	4,500
Very Severe	Over 2.00	Over 10,000	v plus pozzolan[3]	0.45	4,500

1 A lower water-cementitious materials ratio or higher strength may be required for low permeability or for protection against corrosion of embedded items or freezing and thawing (Table 19-A-2).

2 Seawater

3 Pozzolan that has been determined by test or service record to improve sulfate resistance when used in concrete containing Type V cement.

CHEMICAL TEST RESULTS				
SAMPLE #	Conductance (EC) ohms cm	Sulfate Content ppm	Chloride ppm	Cement Type
Soil	1510	140	62	-

COLUMBIA ANALYTICAL SERVICES, INC.

Analytical Report

Client :	Calwest Geotechnical		**Service Request :**	P0700946
Project Name :	DIDM		**Date Collected :**	10/10/07
Project Number :	G4972		**Date Received :**	10/12/07
Sample Matrix :	SOIL			

Inorganic Parameters

Sample Name : DIDM
Lab Code : P0700946-001 Basis : Wet
Test Notes :

Analyte	Units	Analysis Method	MRL	Dilution Factor	Date/Time Analyzed	Date Extracted	Result	Result Notes
Chloride	mg/Kg (ppm)	300.0	10	1	10/16/07 15:13	10/16/07	62	A1
Resistivity	OHMS-cm	9050A	10	1	10/16/07 13:50	10/16/07	1510	
Sulfate	mg/Kg (ppm)	300.0	10	1	10/16/07 15:13	10/16/07	140	A1

A1 Sample preparation: 1:10 (weight:volume) deionized water extraction.

Approved By : Sue Anderson Date : 10/19/07

Report By:DCastillo

APPENDIX D

APPENDIX E

FOUNDATION DESIGN - BEARING CAPACITY

PROJECT: **DIDM Dev. Corp.** **PROJECT NO.:** **4972**

LOCATION: **3635 Elm Ave., Long Beach**

MATERIAL TYPE: NATURAL SOIL

Cohesion {C}:	**360**	PSF
Phi {¢}:	**29**	DEGREES
Blk. Wt. {Gd}:	**125**	PCF

FOOTING TYPE = **1** (PROPOSED)

1) CONTINUOUS
2) SQUARE
3) ROUND
4) RECTANGULAR

FORMULAS USED:

qd= 1.0 x C x Nc + Gd x Df x Nq + 0.5 x B x Gd x Ny

Nq= (e^(Pi x TAN(¢))) x (TAN^2 x (45 + (¢ / 2))) = 16.44

Nc= (Nq - 1) x COT(¢) = 27.86

Ny= (Nq - 1) x TAN(1.4 x ¢) = 13.24

qa= (qd - Gd x Df) / SF { WHERE USUALLY SF = 3 }

FOOTING DIMENSIONS:

WIDTH (B) = **1.00**

DEPTH (Df) = **2.00** { OF FOOTING BELOW LOWEST ADJACENT GROUND SURFACE}

qd = 14967.88 psf **{ BEARING CAPACITY }**	**qa = 4905.96 psf** **{ ALLOWABLE BEARING }**

Reference: C:\OTEMPLTE\SOILS\LAPROG\BEARING.XLW

CAL WEST GEOTECHNICAL

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PRESSURE DISTRIBUTION FOR BRACED WALLS

PROJECT: DIDM

PROJECT NO.: 4972

ANALYSIS: RESTRAINED RETAINING WALL

RETAINING WALL HEIGHT	=	10.00	feet
PHI (ø)	=	29	degrees
COHESION (C)	=	360	psf
UNIT WEIGHT (Gd)	=	125	psf

SOIL TYPE: **TYPE:** 1

1) SAND
2) SOFT TO MEDIUM CLAY
3) STIFF CLAY

USE LOADING DIAGRAM: (A)

No	=	3.5
Ka	=	0.35

@TAN((45 - (Ø/2)²)) }

PRESSURE DISTRIBUTION:

{ SIGMA(h) = 0.65 xKa x Gd x HEIGHT }

SIGMA(h) FOR DIAGRAM (A) = 281.92 PSF

Reference: C:\0TEMPLTE\SOILS\LAPROG\BRACE.XLW

WEST COAST GEOTECHNICAL

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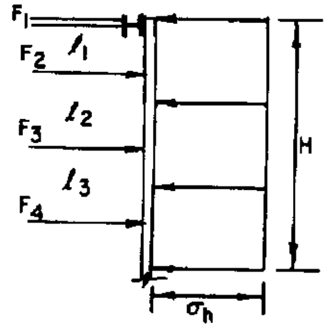
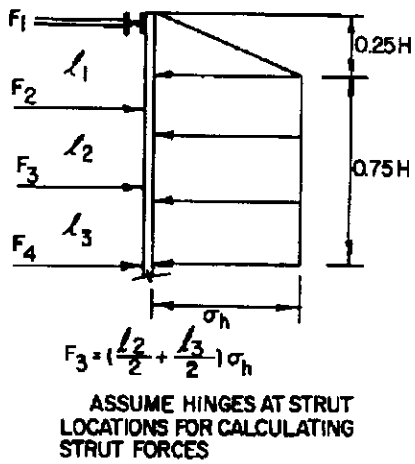


(a) SAND

$\sigma_h = 0.65\ K_A \cdot \gamma H$

WHERE $K_A = TAN^2 (45 - \phi/2)$



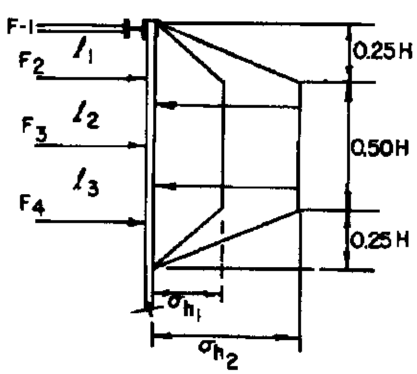
(b) SOFT TO MEDIUM CLAY
(N_o>6)

For clays base the selection on
$N_o = \gamma H/c$

$\sigma_h = K_A \cdot \gamma \cdot H$

$K_A = 1 - m \frac{4C}{\gamma H}$;
m = 1 except where cut is underlain by deep soft normally consolidated clay, then $m = 0.4F_{SB}$

See Figure 28 for Factor of Safety against bottom instability, (F_{SB}): $1 \leq F_{SB} \leq 1.5$



(c) STIFF CLAY
(N_o<4)
For $4<N_o<6$, use larger of diagrams (b) and (c).
$\sigma_{h_1} = 0.2\,\gamma H$; $\sigma_{h_2} = 0.4\,\gamma H$
Use lower value when movements are minimal and short construction period.

FIGURE 26
Pressure Distribution for Brace Loads in Internally Braced Flexible Walls