Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

GEOTECHNICAL ENGINEERING STUDY

PARKERSON PROPERTY

GRAND JUNCTION, COLORADO

Prepared for:

LEGENDS PARTNERS, LLC

MR. RON ABELOE

PROJECT NUMBER: M03032GE

MARCH 13, 2003

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March 13, 2003

Legends Partners LLC
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Attention: Mr. Ron Abeloe

PN: M03032GE

Subject: Geotechnical Engineering Study for the Proposed Parkerson Property Development Grand Junction, Colorado

Mr. Abeloe:

Lambert and Associates is pleased to present our geotechnical engineering study for the subject project. The field study was completed on January 30, 2003. The laboratory study was completed on March 7, 2003. The analysis was performed and the report prepared from March 11, 2003 through March 13, 2003. Our geotechnical engineering report is attached.

We are available to provide material testing services for soil and concrete and provide foundation excavation observations during construction. We recommend that Lambert and Associates, the geotechnical engineer, for the project provide material testing services to maintain continuity between design and construction phases.

If you have any questions concerning the geotechnical engineering aspects of your project please contact us. Thank you for the opportunity to perform this study for you.

Respectfully submitted,

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TABLE OF CONTENTS

M03032GE

TABLE OF CONTENTS
Page Two

1.0 INTRODUCTION

This report presents the results of the geotechnical engineering study we conducted for the proposed Parkerson property site in Grand Junction, Colorado. The study was conducted at the request of Mr. Ron Abeloe, in accordance with our proposal for geotechnical engineering services dated January 30, 2003.

The conclusions, suggestions and recommendations presented in this report are based on the data gathered during our site and laboratory study and on our experience with similar soil conditions. Factual data gathered during the field and laboratory work are summarized in Appendices A and B.

1.1 Proposed Construction

It is our understanding that the Parkerson Property Development will consist of single family residential structures and multi-family structures. We understand that the structures may be single and multi-story wood frame superstructures supported on reinforced concrete foundations. We understand that some basement or other retaining walls may be included in the proposed construction.

1.2 Scope of Services

Our services included geotechnical engineering field and laboratory studies, analysis of the acquired data and report preparation for the proposed site. The scope of our services is outlined below.

- The field study consisted of describing and sampling the soil materials encountered in twenty-five (25) small diameter continuous flight auger advanced test borings at the proposed subdivision location.

- The materials encountered in the test borings were described and samples retrieved for the subsequent laboratory study.

- The laboratory study included tests of select soil samples obtained during the field study to help assess:

. the soil strength potential (internal friction angle and cohesion) of samples tested,
. the swell and expansion potential of the samples tested,
. the settlement/consolidation potential of the samples tested,
. the moisture content and density of samples tested
. moisture content-dry density relationship of select soil subgrade samples,
. subgrade support characteristics of select subgrade samples and
. the soil sulfate concentration of soil samples tested.

- This report presents our geotechnical engineering comments, suggestions and recommendations for planning and design of site development including:

. viable foundation types for the conditions encountered,
. allowable bearing pressures for the foundation types,
. lateral earth pressure recommendations for design of laterally loaded walls, and
. geotechnical engineering considerations and recommendations for concrete slab on grade floors.

- Our comments, suggestions and recommendations are based on the subsurface soil and ground water conditions encountered during our site and laboratory studies.

- Our study did not include any environmental or geologic hazard issues.

2.0 SITE CHARACTERISTICS

Site characteristics include observed existing and pre-existing site conditions that may influence the geotechnical engineering aspects of the proposed site development.

2.1 Site Location

The proposed development site is located south of Patterson Road and east of 28-1/2 road. The proposed development is located adjacent to the east of the existing Legends Subdivision in Grand Junction, Colorado. A project vicinity map is presented on Figure 1.

2.2 Site Conditions

At the time of our field study the proposed development site was vacant. The proposed development site slopes down to the south at an inclination of about ten (10) to one (1) (horizontal to vertical) and flatter. Man placed fill material is located on the majority of the proposed development. The man placed fill material consisting generally of clay soil material with some scattered construction debris. The site contained sparse cover of dryland type vegetation.

2.3 Subsurface Conditions

The subsurface exploration consisted of observing, describing and sampling the soil materials encountered in twenty-five (25) auger advanced test borings. The approximate locations of the test borings are shown on Figure 2. The logs describing the soil materials encountered in the test borings are presented in Appendix A.

The materials encountered in the test borings consisted generally of silty clay material with varying amounts of sand and gravel to a depth of about one (1) to ten (10) feet. The silty clay soils tested have a moderate to high swell potential when wetted and may consolidate under light loading conditions.

Man placed fill was encountered in some of the test borings and was observed stockpiled on the proposed development site. The man placed fill appeared to consist generally of silty and sandy clay with construction debris. We anticipate that the existing man placed fill is of poor quality.

Formational material was encountered in the test borings at a depth of about one (1) to ten (10) feet. The formational material was a silty clay shale of the Mancos Shale formation. The formational shale material tested has a high to very high swell potential when wetted. The formational shale encountered in the test borings had very degrees of weathering at shallow depths.

No free subsurface water was encountered in the test borings at the time of our field study. We anticipate that the free subsurface water elevation may fluctuate with seasonal and other varying conditions. Our experience in the area indicates that fractured layers may exist in the formational material and the fractured layers may carry or store water.

At the time of our field study the proposed development site was not irrigated. It has been our experience that after the site is developed and once landscape irrigation begins the free subsurface water level may tend to rise. In some cases the free subsurface water level rise, as a result of landscape irrigation and other development influences, can be fairly dramatic and the water level may become very shallow.

It is difficult to predict if unexpected subsurface conditions will be encountered during construction. Since such conditions may be found, we suggest that the owner and the contractor make provisions in their budget and construction schedule to accommodate unexpected subsurface conditions.

2.4 Seismic Consideration

Labeled faults near the site are 71Q, 72Q, 73, 81Q, 82Q, 83Q, 84Q, 85Q, 86Q, 87Q, 88Q, 89Q and 179Q, approximately twenty (20) to twenty five (25) miles north, west and south of the site. The fault labels are from Colorado Geological Survey Bulletin 43, "Earthquake Potential in Colorado". The fault number is followed by letters, the letters signify the oldest and youngest units displaced by the fault, or in the case of only one letter, the most recent movement.

The labeled faults are associated with the Uncompahgre block uplift and have displaced Quaternary period geologic units. (Kirkham, Rogers, 1981). The location of the faults is presented on Figure 3.

Based on Bulletin 43 Grand Junction is located in the Colorado Plateau Geologic Province. The maximum credible earthquake estimated by Bulletin 43 for the Colorado Province is a magnitude of 5.5 to 6.5. Spectral analysis for earthquakes presented in "Site-Dependent Spectra for Earthquake-Resistant Design" by H. Bolton Seed, Celso Ugas, and John Lysmer, February 1976 indicates that for an earthquake magnitude of 6 ½ at a distance of twenty (20) miles from the site with a spectral damping of 5 percent for stiff site conditions results in a spectral acceleration of 0.35g with the respective site period estimated between 1 and 3 seconds may be appropriate representation for site conditions.

Based on the subsurface conditions encountered and the 2000 Uniform building code we suggest you consider using UBC site class C for UBC normalized response spectra shape. Additional site geophysical studies would be required to verify our assumptions used in our site seismic assessment. We are available to discuss this with you.

3.0 ON-SITE DEVELOPMENT CONSIDERATIONS

We anticipate that the subsurface water elevation may fluctuate with seasonal and other varying conditions. Deep excavations may encounter soils that tend to cave or a possibility of subsurface water. Our experience in the area indicates that fractured layers may exist in the formational material and that the fractured layers may carry or store water. If water is encountered, it may be necessary to dewater construction excavations to provide more suitable working conditions. Excavations should be well braced or sloped to prevent wall collapse. Federal, state and local safety codes should be observed. All construction excavations should conform to Occupational Safety and Health Administration (OSHA) standards or safer.

The site construction surface should be graded to drain surface water away from the site excavations. Surface water should not be allowed to accumulate in excavations during construction. Accumulated water could negatively influence the site soil conditions. Construction surface drainage should include swales, if necessary to divert surface water away from the construction excavations.

Man placed fill material exists on site. The quality of the man placed fill is not known and may not be suitable for support of the structure or structural components. The quality of the existing man placed fill should be verified or the fill removed and replaced with compacted structural fill prior to supporting building or building components on the fill.

The formational material encountered in the test borings was very hard. We anticipate that it may be possible to excavate this material; however, additional effort may be necessary. We do not recommend blasting to aid in excavation of the material. Blasting may fracture the formational material which will reduce the support characteristic integrity of the formational material.

It has been our experience that sites in developed areas may contain existing subterranean structures or poor quality man placed fill. If subterranean structures or poor quality man placed fill are suspected or encountered, they should be removed and replaced with compacted structural fill as discussed under COMPACTED STRUCTURAL FILL below.

The soil materials exposed in the bottom of the excavation may be very moist and may become yielding under construction traffic during construction. It may be necessary to use techniques for placement of fill material or foundation concrete which limits construction traffic in the vicinity of the very moist soil material. If yielding should occur during construction it may be necessary to construct a subgrade stabilization fill blanket or similar to provide construction traffic access. The subgrade stabilization blanket may include over excavating the subgrade soils one (1) to several feet and replacing with aggregate subbase course type material. The stabilization blanket may also include geotextile stabilization fabric at the bottom of the excavation prior to placement of aggregate subbase course stabilization fill.

Other subgrade stabilization techniques may be available. We are available to discuss this with you.

4.0 FOUNDATION RECOMMENDATIONS

Geotechnical engineering considerations which influence the foundation design and construction recommendations presented below are discussed in Appendix C.

We have analyzed drilled piers and spread footings as potential foundation systems for the proposed structure. These are discussed below. Due to the number of possible foundation types available and design and construction techniques there may be design alternatives which we have not presented in this report. We are available to discuss other foundation types. We have provided design parameters for several foundation types. Of these, because of the expansion potential of the site soils, we feel that drilled piers will provide the foundation type with the least likelihood of significant post construction movement.

We recommend that the entire structure be supported on only one foundation type. Combining foundation types will result in differential and unpredictable foundation performance between the varying foundation types.

All of the design parameters presented below are based on techniques performed by an experienced competent contractor, high quality craftsmanship and care during construction. We recommend post construction cognizance of the potential swelling soil hazard with appropriate post construction maintenance. The spread footing recommendations include recommended design and construction techniques to reduce the influence of movement of the swelling soil materials supporting the foundation but should not be interpreted as solutions for completely mitigating the potential for movement from swelling soil supporting footings.

Because of site configuration and planned construction you may decide that it is not practical to support the structure on drilled piers. For this reason we have provided spread footing recommendations as an alternative foundation to drilled piers. The spread footing recommendations include recommended design and construction techniques to reduce the influence of swelling soils supporting the foundation but should not be interpreted as solutions for completely mitigating the potential for movement from swelling soil supporting footings.

Exterior column supports should be supported by foundations incorporated into the foundation system of the structure not supported on flatwork. Column supports placed on exterior concrete flatwork may move if the support soils below the concrete slab on grade become wetted and swell or freeze and raise or settle. Differential movement of the exterior columns may cause stress to accumulate in the supported structure and translate into other portions of the structure.

4.1 Drilled Piers

Drilled piers or caissons that are drilled into the unweathered formational material may be used to support the proposed structure. The piers should be drilled into the formational material a distance equal to at least two (2) pier diameters, or minimum of five (5) feet into the hard unweathered formational material, whichever is deeper. The piers should be designed as end bearing piers using a formational material bearing capacity of 20,000 pounds per square foot and a side friction of 2,000 pounds per square foot for the portion of the pier in the unweathered formational material. The drilled piers should be designed with a minimum dead load of 5,000 pounds per square foot. Varying weathering and formational competence may result in a shorter required penetration of the drilled piers into the formational material to provide the end bearing capacity discussed above. We should be contacted to observe the pier drilling operations and provide additional geotechnical engineering suggestions and recommendations for design bearing capacity and minimum penetration into the formational material as needed.

There are differing theories on the use of side shear as part of the load carrying assessment of drilled pier foundation systems. The differences are related to the strain compatibility between end bearing and side shear. One theory is that mobilization of the drilled pier is required to generate the side shear soil strength values. This mobilization would require the movement of the bottom of the pier which may not be a desirable characteristic. Another theory is that the support materials will develop static frictional forces in contact with the materials along the surface of the pier.

It is our opinion that sufficient movement of the piers to mobilize skin friction for bearing support may result in undesirable performance of the pier in the form of settlement. We suggest consideration to the amount of settlement tolerable to the structure be included in your assessment if skin friction is used in your design as part of the bearing support of the drilled pier.

We suggest that piers be designed using end bearing capacity only. The side shear in the formational material may be used for the design to resist uplift forces. When using skin friction for resisting uplift we suggest that you discount the upper portion of the pier embedment in the formational material to a depth of at least one and one-half (1 1/2) pier diameters into the formational material.

The bottom of the pier holes should be thoroughly cleaned to insure that all loose and disturbed materials are removed prior to placing pier concrete. It is very important to thoroughly clean the bottom of the pier holes prior to placement of the pier concrete. Loose disturbed material left in the bottom of the pier hole will likely result in long term settlement of the piers as the disturbed material consolidated under the pier loads. The pier holes should be observed during the excavation and cleaning operation and again immediately prior to placement of pier concrete after steel reinforcement and any casing materials have been installed to verify that material was not dislodge into the pier hole during steel reinforcement or casing placement.

Because of the rebounding potential in the formational materials when unloaded by excavation and because of the possibility of desiccation of the newly exposed material we suggest that concrete be placed in the pier holes immediately after excavation and cleaning.

If the piers are designed and constructed as discussed above we anticipate that the post construction settlement potential of each pier may be less than about one quarter (1/4) inch.

The portion of the pier above the formational surface and in the weathered formational material should be cased with a sono tube or similar casing to help prevent flaring on the top of the pier holes and help provide a positive separation of the pier concrete and the adjacent soils.

Construction of the piers should include extreme care to prevent flaring of the top of the piers. Enlarged portions of the drilled pier excavation near the surface may perform similar to the top flaring. Preventing flaring may be aided by casing the drilled pier excavation with a sono tube or similar casing. Reducing flaring is to help reduce the potential of swelling soils to impose uplift forces which will put the pier in tension. The drilled piers should be vertically reinforced to provide tensile strength in the piers should swelling on site soils apply tensile forces on the piers. The structural engineer should be consulted to provide structural design recommendations.

Free ground water was not encountered in the test borings at the time of the field study. Our experience in the area indicates that fractured layers may exist in the formational material and that the fractured layers may carry or store water. If ground water is encountered, the pier holes should be dewatered prior to placing pier concrete. No pier concrete should be placed when more than six (6) inches of water exists in the bottom of the pier holes. The piers should be filled with a tremie placed concrete immediately after the drilling and cleaning operation is complete.

Caving soil materials were encountered in our test borings. It may be necessary to case the pier holes with temporary casing to prevent caving during pier construction. If drilled piers are considered as a viable foundation system for the proposed structure the owner should be aware of potential difficulties that may occur during the drilling of the pier holes. Drilling pier holes in soil materials that tend to cave may not be possible with the drill rigs locally available to the area. Drilled pier foundations may require special considerations during the design and scheduling of construction.

Difficult drilling conditions were encountered with our drill rig during our field study. We anticipate that pier drilling equipment available in the area may have difficulty drilling the formational material. It may be necessary to obtain specialty drilling equipment, possibly not available in western Colorado, to advance the drilled pier holes. We are available to discuss this with you.

The contact between the weathered formational material and the unweathered formational material may be gradual and difficult to identify. The minimum penetration of the drilled pier into the unweathered formational material as discussed above is important for the long term performance of the pier foundation. We should be contacted to observe the pier drilling operation to verify the construction techniques used, the material encountered during the drilling operation and condition of the bottom of the drilled pier hole prior to placement of pier concrete.

The structural engineer should be consulted to provide structural design recommendations for the drilled piers and grade beam foundation system.

4.2 Spread Footings

In our analysis it was necessary to assume that the material encountered in the test borings extended throughout the building site and to a depth below the maximum depth of the influence of the foundations. We should be contacted to observe the soils exposed in the foundation excavations prior to placement of foundations to verify the assumptions made during our analysis.

We anticipate that the surface of the formational material may undulate which may result in a portion of the footings supported on the overlying soils and a portion of the foundation members supported on the formational material. If this happens the foundations will perform differently between the areas supported on formational material and the areas supported on the non-formational material. For this reason we suggest that if formational material is encountered only in portions of the foundation excavations at footing depth the foundation in all areas should be extended to support all foundation members on the formational material or the footings should be supported entirely on a blanket of compacted structural fill which is supported by the formational material.

The bottom of the foundation excavations should be thoroughly cleaned and observed when excavated. Any loose or disturbed material exposed in the foundation excavation should be removed prior to placing foundation concrete.

The bottom of the foundation excavations should be compacted prior to placing compacted structural fill or foundation concrete. We suggest the materials exposed be compacted to at least ninety (90) percent of the materials moisture content-dry density relationship (Proctor) test, ASTM D1557. Excavation compaction is to help reduce the influence of any disturbance that may occur during the excavation operations. Any areas of loose, low density or yielding soils evidenced during the excavation compaction operation should be removed and replaced with compacted structural fill. Caution should be exercised during the excavation compaction operations. Excess rolling or compacting may increase pore pressure of the subgrade soil material and degrade the integrity of the support soils. Loose or disturbed material in the bottom of the foundation excavations which are intended to support structural members will likely result in large and unpredictable amounts of settlement, if the loose or disturbed material is not compacted .

The bottom of any footings exposed to freezing temperatures should be placed below the maximum depth of frost penetration for the area. Refer to the local building code for details.

All footings should be appropriately proportioned to reduce the post construction differential settlement. Footings for large localized loads should be designed for bearing pressures and footing dimensions in the range of adjacent footings to reduce the potential for differential settlement. We are available to discuss this with you.

Foundation walls may be reinforced for geotechnical engineering purposes. We suggest at least two (2) number 5 bars, continuous at the top and the bottom (4 bars total), at maximum vertical spacing. This will help provide the walls with additional beam strength and help reduce the effects of slight differential settlement. The walls may need additional reinforcing steel for structural purposes. The structural engineer should be consulted for foundation design. The structural engineering reinforcing design tailored for this project will be more appropriate than the suggestions presented above.

The structure may be founded on spread footings. Spread footings may be placed either on the natural undisturbed soils or on a blanket of compacted structural fill. The blanket of compacted structural fill is to help provide uniform support for the footings, to help mask swelling soils supporting the footings and to help reduce the anticipated post construction settlement. The anticipated post construction settlement and associated fill thickness supporting the footings are presented below. We suggest that you consider the foundation be supported on a blanket of compacted structural fill at least two (2) feet thick to help mask the influence of swelling soils supporting the footings. The blanket of compacted structural fill will not prevent movement of the footings from swelling soils but will mask the influence of volume changes of the soils supporting the footings. If the footings are supported on a blanket of compacted structural fill the blanket of compacted structural fill should extend beyond each edge of each footing a distance at least equal to the fill thickness. This concept is shown on Figure 4. Geotechnical engineering recommendations for constructing compacted structural fill are presented below.

A blanket of compacted fill will help to mask the influence of the very high measured swell pressures when the foundation soils become wetted. If you choose spread footings we strongly recommend high design dead load, deep embedment and a thick fill blanket.

The bearing capacity will depend on the minimum depth of embedment of the bottom of the footings below the lowest adjacent grade and the support characteristics of the soils supporting the foundation. Other characteristics may influence embedment. The embedment concept is shown on Figure 5. The bearing capacity will depend on the type of material supporting the foundation. Bearing capacity for material types supporting the foundation and associated minimum depth of embedment of the bottom of the footing below the lowest adjacent grade are presented below.

SPREAD FOOTING SOIL BEARING CAPACITY CONTINUOUS (POUNDS PER SQUARE FOOT)	ISOLATED	A* (feet)
1,000	1,250	0
1,250	1,500	1
1,500	1,750	2
1,750	2,000	3

SPREAD FOOTING FORMATIONAL MATERIAL BEARING CAPACITY CONTINUOUS (POUNDS PER SQUARE FOOT)	ISOLATED	A* (feet)
3,000	3,500	0
3,500	4,000	1
4,000	4,500	2
4,500	5,000	3

A* Minimum depth of embedment for footings adjacent to level areas.

If deeper embedment is considered for increased bearing capacity greater than presented above, we should be contacted to provide additional analysis and recommendations as needed. The bearing capacity design value is based on several considerations and these may change with depth.

The bearing capacity may be increased by about twenty (20) percent for transient loads such as wind and seismic loads.

It is our opinion that footings should have a minimum depth of embedment of at least one (1) foot on all sides to provide a more predictable long term performance of the footing. We understand that construction techniques typically used in the area may result in some of the footings in the crawl space constructed without significant embedment of the bottom of the footing below the lowest adjacent grade. For this reason we have provided design values for footings constructed with little or no embedment. It is our opinion that the performance of footing constructed without embedment may be influenced by erosion, temperature changes, moisture content changes, swell potential of the soil supporting the footings and weathering of the soils supporting the footings and will have a less predictable settlement response than footings with embedment.

Exterior footings and footings with uneven backfill may result in movement of the footings. Embedment of the footings on all sides will help reduce the potential for movement of footings with uneven backfill. We do not recommend exterior footings or footings with uneven backfill be constructed without a minimum depth of embedment of the bottom of the footing below the lowest adjacent grade of at least one (1) foot on all sides of the footings.

The soil sample tested had a measured swell pressure ranging from about 300 to about 2,600 pounds per square foot. When wetted the site soil materials have the ability to raise supported foundation members with loads less than the swell pressure. The foundation design should be as rigid as possible with as high of a dead load as can be available. The greater the dead load on the footings the less the potential for movement from the foundation soils should they become wetted. If the soils become wetted they will swell and will raise the foundation portions supported on the wetted soils. If the structure is supported on spread footings the owner must realize that post construction movement of the footings is likely. We are available to discuss the implications of supporting foundations on swelling soils.

Interior column loads supported on spread footings which are structurally connected to the other foundation members will provide more uniform performance of the interior footings with respect to the other foundation members and will help reduce the potential differential settlement between interior and exterior foundation members. The foundation walls should be designed to act as beams to distribute stresses associated with the swelling soils. The beam design should be addressed by the project structural engineer.

Exterior column supports should be supported by foundations incorporated into the foundation system of the structure not supported on flatwork. Column supports placed on exterior concrete flatwork may move if the support soils below the concrete slab on grade become wetted and swell or freeze and raise or settle. Differential movement of the exterior columns may cause stress to accumulate in the supported structure and translate into other portions of the structure.

The estimated post construction settlement and swell potential may be reduced by placing the footings on a blanket of compacted structural fill. The estimated post construction settlement and associated thickness of compacted structural fill are presented below.

THICKNESS OF COMPACTED STRUCTURAL FILL SUPPORTING FOOTINGS	ESTIMATED POST CONSTRUCTION SETTLEMENT (INCHES)
0	About 1
*B/2	About ¾
B	About ½

*B is equal to the footing width

The settlement values above are appropriate for footings with a width of about two (2) feet or less. Larger footings should be analyzed on a footing, load and width specific basis.

The calculated settlement estimates are theoretical only. Actual settlement could vary throughout the site and with time.

If the footings are supported on a blanket of compacted structural fill, the blanket of compacted structural fill should extend beyond each edge of each footing a distance at least equal to the fill thickness. This concept is shown on Figure 4. Compacted Structural Fill is discussed in section 6.0 below.

The site soil samples tested have a measured swell pressure up to about 2,600 pounds per square foot. This swell pressure was measured for soils at the initial moisture content of the soil sample tested. The measured swell pressure may be influenced by disturbance of the sample during the sampling operation and the soil suction potential.

Changes in the initial moisture content will significantly influence the swell pressure of the site soils. If the initial moisture content of the foundation soils is less than that of the test sample the actual swell pressures will likely be significantly higher than measured. If the initial moisture content of the foundation soils is greater than that of the test sample the actual swell pressures may be less than measured.

Our experience with the clay soils and formational material in the area indicates that the in situ swell pressure of the clay soils may be much higher than measured. Should the site soils become wetted after construction they may develop this swell pressure which will cause movement of the influenced structure components. Much of the anticipated soil volume increase would cause differential movement across the structure foundation which could cause structural damage. Due to the potential for movement we do not feel that the conventional spread footing foundation system is the best option for the foundation design.

The bottom of the foundation excavations should be thoroughly cleaned and observed by the project Geotechnical Engineer or his representative when excavated. Any loose or disturbed material exposed in the foundation excavation should be removed or remedied prior to additional construction.

We recommend that we be contacted to observe the foundation excavations and backfill operations during construction to verify the soil support conditions and our assumptions upon which our recommendations are based. If necessary we may revise our recommendations based on our observations. We are available to provide material testing services during the construction phase of the project.

5.0 INTERIOR FLOOR SLAB DISCUSSION

It is our understanding that, as currently planned, concrete slab on grade floors may be included in the proposed structures. We understand that concrete slab on grade garage floors may be included in the construction. The geotechnical engineering suggestions and recommendations for interior floor slabs presented below are appropriate for garage floor slabs. The natural soils that will support interior floor slabs are stable at their natural moisture content. However, the owner should realize that when wetted, the site soils may experience volume changes. The site soil samples tested had a measured swell pressure ranging from about 300 to about 2,600 pounds per square foot and an associated magnitude of 0.1 to about 8.0 percent of the wetted soil volume at a surcharge load of 100 pounds per square foot.

Conditions which vary from those encountered during our field study may become apparent during excavation. We should be contacted to observe the conditions exposed at concrete slab on grade subgrade elevation to verify the assumptions made during the preparation of this report and to provide additional geotechnical engineering suggestions and recommendations as needed.

Engineering design dealing with swelling soils is an art which is still developing. The owner is cautioned that the soils on this site may have swelling potential and concrete slab on grade floors and other lightly loaded members may experience movement when the supporting soils become wetted. We suggest you consider floors suspended from the foundation systems as structural floors or a similar design that will not be influenced by subgrade volume changes. If the owner is willing to accept the risk of possible damage from swelling soils supporting concrete slab on grade floors, the following recommendations to help reduce the damage from swelling soils should be followed. These recommendations are based on generally accepted design and construction procedures for construction on soils that tend to experience volume changes when wetted and are intended to help reduce the damage caused by swelling soils. Lambert and Associates does not intend that the owner, or the owner's consultants should interpret these recommendations as a solution to the problems of swelling soils, but as measures to reduce the influence of swelling soils.

Concrete flatwork, such as concrete slab on grade floors, should be underlain by compacted structural fill. The layer of compacted fill should be at least two (2) feet thick or thicker and constructed as discussed under COMPACTED STRUCTURAL FILL below. A two (2) feet thick blanket of structural fill material beneath the concrete flatwork is not sufficient to entirely mask the settlement or swell potential of the subgrade soil material but will only provide better subgrade conditions for construction.

The natural soil materials exposed in the areas supporting concrete slab on grade floors should be kept very moist during construction prior to placement of concrete slab on grade floors. This is to help increase the moisture regime of the potentially expansive soils supporting floor slabs and help reduce the expansion potential of the soils. We are available to discuss this concept with you.

Concrete slab on grade floors should be provided with a positive separation, such as a slip joint, from all bearing members and utility lines to allow their independent movements and to help reduce possible damage that could be caused by movement of soils supporting interior slabs. The floor slab should be constructed as a floating slab. All water and sewer pipe lines should be isolated from the slab. Any equipment placed on the floating floor slab should be constructed with flexible joints to accommodate future movement of the floor slab with respect to the structure. We suggest partitions constructed on the concrete slab on grade floors be provided with a void space above or below the partitions to relieve stresses induced by elevation changes in the floor slab.

The concrete slabs should be scored or jointed to help define the locations of any cracking. We recommend that joint spacing be designed as outlined in ACI 224R. In addition joints should be scored in the floors a distance of about three (3) feet from, and parallel to, the walls.

It should be noted that when curing fresh concrete experiences shrinkage. This shrinkage almost always results in some cracks in the finished concrete. The actual shrinkage depends on the configuration and strength of the concrete and placing and finishing techniques. The recommended joints discussed above are intended to help define the location of the cracks but should not be interpreted as a solution to shrinkage cracks. The owner must understand that concrete flatwork will contain shrinkage cracks after curing and that all of the shrinkage cracks may not be located in control joints. Some cracking at random locations may occur.

If moisture migration through the concrete slab on grade floors will adversely influence the performance of the floor or floor coverings we suggest that a moisture barrier may be installed beneath the floor slab to help discourage capillary and vapor moisture rise through the floor slab. The moisture barrier may consist of a heavy plastic membrane, six (6) mil or greater, protected on the top and bottom by clean sand. The clean sand will help to protect the plastic from puncture. The layer of clean sand on the top of the plastic membrane will help the overlying concrete slab cure properly. According to the American Concrete Institute, proper curing requires at least three (3) to six (6) inches of clean sand between the plastic membrane and the bottom of the concrete. The plastic membrane should be lapped and taped or glued and protected from punctures during construction.

The Portland Cement Association suggests that welded wire reinforcing mesh is not necessary in concrete slab on grade floors when properly jointed. It is our opinion that welded wire mesh may help improve the integrity of the slab on grade floors. We suggest that concrete slab on grade floors should be reinforced, for geotechnical purposes, with at least 6 x 6 - W2.9 x W2.9 (6 x 6 - 6 x 6) welded wire mesh positioned midway in the slab. The structural engineer should be contacted for structural design of floor slabs.

6.0 COMPACTED STRUCTURAL FILL

Material characteristics desirable for compacted structural fill are discussed in Appendix C. Areas that are over excavated or slightly below grade should be backfilled to grade with properly compacted structural fill or concrete, not loose fill material. If backfilled with other than compacted structural fill material or concrete there will be significant post construction settlement proportional to the amount of loose material.

The natural on site soils are not suitable for use as compacted structural fill material supporting building or structure members because of their clay content and swell potential. The natural on-site soils may be used as compacted fill in areas that will not influence the structure such as to establish general site grade. We are available to discuss this with you.

All areas to receive compacted structural fill should be properly prepared prior to fill placement. The preparation should include removal of all organic or deleterious material. The areas to receive fill material should be compacted after the organic deleterious material has been removed prior to placing the fill material. The area may need to be moisture conditioned for compaction. Any areas of soft, yielding, or low density soil, evidenced during the excavation compaction operation should be removed. The area excavated to receive fill should be moisture conditioned to wet of optimum moisture content as part of the preparation to receive fill. Fill should be moisture conditioned, placed in thin lifts not exceeding six (6) inches in compacted thickness and compacted to at least ninety (90) percent of maximum dry density as defined by ASTM D1557, modified moisture content-dry density (Proctor) test.

After placement of the structural fill the surface should not be allowed to dry prior to placing concrete or additional fill material. This may be achieved by periodically moistening the surface of the compacted structural fill as needed to prevent drying of the structural fill. We are available to discuss this with you.

The soil materials exposed in the bottom of the excavation may be very moist and may become yielding under construction traffic during construction. It may be necessary to use techniques for placement of fill materials or foundation concrete which limit construction traffic in the very moist soil materials. If yielding should occur during construction it may be necessary to construct a subgrade stabilization fill blanket or similar to provide construction traffic access. We are available to discuss this with you.

We recommend that the geotechnical engineer or his representative be present during the excavation compaction and fill placement operations to observe and test the material.

7.0 LATERAL EARTH PRESSURES

Laterally loaded walls supporting soil, such as basement walls, will act as retaining walls and should be designed as such. Walls that are designed to deflect and mobilize the internal soil strength should be designed for active earth pressures. Walls that are restrained so that they are not able to deflect to mobilize internal soil strength should be designed for at-rest earth pressures. The values for the lateral earth pressures will depend on the type of soil retained by the wall, backfill configuration and construction technique. If the backfill is not compacted the lateral earth pressures will be very different from those noted below.

Lateral earth pressure (L.E.P.) values are presented below:

	Level Backfill with on-site soils (pounds per cubic foot per foot of depth)
Active L.E.P.	60
At-rest L.E.P.	80
Passive L.E.P.	250

The soil samples tested have measured swell pressure of about 300 to about 2,600 pounds per square foot. Our experience has shown that the actual swell pressure may be much higher. If the retained soils should become moistened after construction the soil may swell against retaining walls. The walls should be designed to resist the swell pressure of the soils.

The above lateral earth pressures may be reduced by overexcavating the wall backfill area beyond the zone of influence and backfilling with crushed rock type material. The zone of influence concept is presented on Figure 6.

The lateral earth pressure design parameters may change significantly if the area near the wall is loaded or surcharged or is sloped. If any of these conditions occur we should be contacted for additional design parameters tailored to the specific site and structure conditions.

Suggested lateral earth pressure (L.E.P.) values if the backfill is overexcavated beyond the zone of influence and backfilled with crushed rock are presented below.

	Level Backfill with crushed rock material (pounds per cubic foot per foot of depth)
Active L.E.P	35
At-rest L.E.P.	50

If the area behind a wall retaining soil material is sloped we should be contacted to provide lateral earth pressure design values tailored for the site specific sloped conditions.

Resistant forces used in the design of the walls will depend on the type of soil that tends to resist movement. We suggest that you consider a coefficient of friction of 0.25 for the on site soil.

The lateral earth pressure values provided above, for design purposes, should be treated as equivalent fluid pressures. The lateral earth pressures provided above are for level well drained backfill and do not include surcharge loads or additional loading as a result of compaction of the backfill. Unlevel or non-horizontal backfill either in front of or behind walls retaining soils will significantly influence the lateral earth pressure values. Care should be taken during construction to prevent construction and backfill techniques from overstressing the walls retaining soils. Backfill should be placed in thin lifts and compacted, as discussed in this report to realize the lateral earth pressure values.

Walls retaining soil should be designed and constructed so that hydrostatic pressure will not accumulate or will not affect the integrity of the walls. Drainage plans should include a subdrain behind the wall at the bottom of the backfill to provide positive drainage. Exterior retaining walls should be provided with perimeter drain or weep holes to help provide an outlet for collected water behind the wall. The ground surface adjacent to the wall should be sloped to permit rapid drainage of rain, snow melt and irrigation water away from the wall backfill. Sprinkler systems should not be installed directly adjacent to retaining or basement walls.

8.0 DRAIN SYSTEM

A drain system should be provided around building spaces below the finished grade and behind any walls retaining soil. The drain systems are to help reduce the potential for hydrostatic pressure to develop behind retaining walls. A sketch of the drain system is shown on Figure 7.

Subdrains should consist of a three (3) or four (4) inch diameter perforated rigid pipe surrounded by a filter. The filter should consist of a filter fabric or a graded material such as washed concrete sand or pea gravel. If sand or gravel is chosen the pipe should be placed in the middle of about four (4) cubic feet of aggregate per linear foot of pipe. The drain system should be sloped to positive gravity outlets. If the drains are daylighted the drains should be provided with all weather outlets and the outlets should be maintained to prevent them from being plugged or frozen. We do not recommend that the drains be discharged to dry well type structures. Dry well structures may tend to fail if the surrounding soil material becomes wetted and swells or if the ground water rises to a elevation of or above the discharge elevation in the dry well. We should be called to observe the soil exposed in the excavations and to verify the details of the drain system.

A drain blanket may be constructed beneath the basement concrete slab on grade floor slab to intercept water that may tend to rise into the basement area. The drain blanket should be at least one foot thick and consist of a free draining sand or gravel material which is compacted as discussed under Compacted Structural Fill above, section 6.0 The subgrade below the drain blanket should be sloped to collection points prior to constructing the drain blanket. A perforated pipe should be installed at the collection points and graded to discharge similar to the foundation drain discussed above. The drain blanket concept is shown on Figure 7. The under slab drain blanket may be considered as part of the structural fill intended to support the floor slab as discussed under Interior Floor Slabs, section 5.0 above. We are available to discuss this concept with you.

9.0 CRAWL SPACE CONSIDERATIONS

We anticipate that free subsurface water may be shallow enough during wetter seasons to exist in crawl space areas or create very moist conditions in crawl space areas. We suggest that if it is desired to reduce the influence of water in the crawl space area a foundation drain should be installed as discussed above.

The surface of the crawl space may be provided with a layer of about six (6) inches of clean washed gravel or an impervious geotextile fabric to reduce the inconvenience of very moist or muddy crawl space conditions if these should occur. The crawl space should be adequately vented to reduce the potential for humidity to accumulate in the crawl space area.

10.0 PAVEMENT SECTION DESIGN RECOMMENDATIONS

It is our understanding that the proposed development will include paved roadways and parking areas. The paved areas will include asphalt paved parking areas, concrete paved aprons and concrete sidewalks. Our pavement section analysis was based on estimated traffic volumes, laboratory test results of the soils sampled during our field study, and on our experience on similar projects. The traffic volume used in our analysis assumed an 18,000 pound equivalent single axle load (ESAL) of 25,000, 50,000, 100,000 and 150,000 repetitions to allow the civil engineer to select the pavement section which most appropriately reflects his anticipate traffic loading included anticipated future growth. Our analysis included pavement sections based on dynamic loading as discussed in the Colorado State Highway Design Manual for Design of Pavement Structures Section 600.

10.1 Subgrade Preparation

Proper performance of the subgrade support soils requires surface preparation, scarification and moisture conditioning, compaction, and surface and subsurface drainage during construction prior to placement of the overlying pavement section materials.

Subgrade preparation may result in areas which yield under construction traffic. If yielding areas are encountered during subgrade preparation in the paved areas, the subgrade material may be overexcavated to a depth of about one foot below the subgrade elevation or more if needed and backfilled with a compacted structural fill. The structural fill material may aid in construction of the paved areas subgrade. The structural fill material should be an aggregate subbase course or aggregate base course type material placed and compacted as discussed below.

All organic and other deleterious material should be removed from the areas proposed for pavement section construction. The soils exposed by the removal of the organic materials should be scarified to a depth of about twelve (12) inches, moisture conditioned to near optimum moisture content, and compacted to at least ninety (90) percent of maximum dry density as defined by ASTM D1557, modified moisture content-dry density relationship (Proctor) test. The moisture conditioning may require addition of water, or air drying if the soil is too moist, in either case, the material should be sufficiently mixed to promote a uniform soil moisture content. The soils should be compacted using machinery designed for soil compaction. Wheel rolling with loaded equipment and other techniques may not provide a uniform, properly compacted roadway subgrade.

Utility trench backfill in areas supporting pavement or other structural components should be placed in thin lifts and compacted to at least ninety (90) percent of the maximum dry density as defined by ASTM D1557 to subgrade elevation.

After the subgrade soils have been prepared the surface should be crowned or surface graded in the same orientation as the proposed final surface of the asphalt pavement. The reason for this is to promote water migration away from the roadway more readily. If the subgrade soil surface is not graded to properly drain, water may accumulate within the pavement section soils. The increased moisture content and subsequent soil strength decrease may promote pavement section support degradation. If a full section asphalt concrete design is used, the subgrade soils should be graded parallel the final asphalt concrete surface for drainage so that a uniform asphalt concrete thickness exists.

10.2 Aggregate Sub-Base and Base Course Material Characteristics and Placement

Specific aggregate types and sources for potential use on the project were not known at the time of the preparation of this report. Our analysis assumed that the proposed aggregate base course would consist of a Class 6 type material, and the aggregate sub-base course would consist of a Class 2 type material, as designated in the "Colorado Department of Highways Standard Specification for Road and Bridge Construction", 1991. If it is desirable to use material which does not meet these criteria we should be contacted to assess the specific material characteristics of the proposed road base and provide additional pavement design sections for differing materials.

The aggregate sub-base and base course materials should be placed on the prepared subgrade soils as soon as possible after the subgrade soils are compacted and graded to drain. Placement of the aggregate materials will help limit the influence of construction and other traffic on the subgrade soil conditions.

The aggregate materials should not be allowed to become segregated either at the source, prior to hauling to the project site, or during the placement of the materials. The coarser aggregate sub-base soils have a greater tendency to become segregated; particularly during the grading and placement operations. Segregated sub-base and base course do not provide as uniform support as well blended materials.

The sub-base and base course materials should be moisture conditioned and compacted to at least ninety-five (95) percent of maximum dry density as defined by ASTM D1557, modified moisture-content-dry density relationship (Proctor) test.

10.3 Asphalt Concrete Materials and Placement

The asphalt concrete should be prepared using a mix design which has been prepared by a professional engineer experienced in asphalt concrete materials. The mix design should establish, as a minimum, the quality of the aggregates used, asphalt concrete material properties, asphalt cement content, mix and lay down temperatures. Either the Marshall Method or Hveem Stabilometer method of mix design may be used for the mix design preparation. We suggest that the asphalt concrete be compacted to at least ninety-five (95) percent of the maximum mix design density.

Aggregate shape maximum size and particle size distribution are important factors influencing the performance of an asphalt concrete mix. Crushed aggregate with fractured faces and angular shapes tend to interlock and provide an asphalt concrete with high strength and limited flexibility. Natural aggregates with rounded shapes tend to provide an asphalt concrete which is more flexible and may have lower strengths than mixes produced with angular shaped aggregates. Incorrect particle or grain size distribution of the aggregate used to manufacture the asphalt concrete can result in poor performance of the in-place asphalt mix. The grain size distribution of the mix aggregate will influence the size and volume of voids and the stability of the asphalt mix. Verification of the asphalt mix design aggregate properties and the asphalt concrete mix should be performed by testing prior to and during the paving operation.

10.4 Flexible Pavement Design Sections

Our laboratory analysis of the support characteristics of the subgrade soils on the project included visual classification and California Bearing Ratio tests. The California Bearing Ratio tests are presented in Appendix B. An "R" value of 5 was used in our analysis. The "R" value was calculated from the California Bearing Ratio test results using "Thickness Design-Asphalt Pavements for Highway and Streets" Manual Series Number 1 by the Asphalt Institute dated September 1981. Alternative pavement sections are presented below. The pavement thickness sections below are based on the serviceability index 2.5 nomograph as recommended in the Colorado Department of Transportation Highway Design Manual for Streets with an ADT (Average Daily Traffic) greater than 750 and traffic volume as discussed above.

Construction traffic will have a greater influence on the performance of the pavement section than the residential use after construction. The design recommendations presented below are based on typical post construction residential use and do not include accommodation for heavy loading as a result of construction traffic. It may be beneficial to consider partial pavement section construction for use during on-site development construction with the section repaired and completed after the heavy construction traffic use has ended. This technique may provide a more serviceable and structurally acceptable pavement for the completed project.

PAVEMENT THICKNESS DESIGN SECTIONS
*ESAL = 25,000

Asphalt Concrete (inches)	Aggregate Base Course Class 6 or Similar (inches)	Aggregate Subbase Course Class 2 or Similar (inches)	Reconditioned Subgrade (Inches)
2	4½	15	12
2	14	0	12
3	4½	10	12
3	11	0	12
6½	0	0	12

PAVEMENT THICKNESS DESIGN SECTIONS
*ESAL = 50,000

Asphalt Concrete (inches)	Aggregate Base Course Class 6 or Similar (inches)	Aggregate Subbase Course Class 2 or Similar (inches)	Reconditioned Subgrade (Inches)
2½	5	15	12
2½	14	0	12
3	5	12	12
3	13	0	12
6¾	0	0	12

PAVEMENT THICKNESS DESIGN SECTIONS
*ESAL = 100,000

Asphalt Concrete (inches)	Aggregate Base Course Class 6 or Similar (inches)	Aggregate Subbase Course Class 2 or Similar (inches)	Reconditioned Subgrade (Inches)
3	5½	15	12
3	15	0	12
4	5½	10	12
4	12	0	12
7½	0	0	12

PAVEMENT THICKNESS DESIGN SECTIONS
*ESAL = 150,000

Asphalt Concrete (inches)	Aggregate Base Course Class 6 or Similar (inches)	Aggregate Subbase Course Class 2 or Similar (inches)	Reconditioned Subgrade (Inches)
3	6	18	12
3	17	0	12
4	6	12	12
4	14	0	12
8	0	0	12

* Equivalent 18,000 pounds single axle load

Pavement thickness design section of less than three (3) inches of asphalt over aggregate base course may be used, although, because of the shorter life before maintenance and the relatively poor long term performance, we suggest that this be considered as an intermediate design section only. If a lesser design section is used we suggest you consider a later asphalt overlay of appropriate thickness to extend the life of the pavement section. The overlay should be constructed prior to any visible distress occurring in the pavement.

The asphalt concrete pavement should be placed on the prepared support section as soon as possible so that interim traffic does not decrease the integrity of the support section.

10.5 Rigid Pavement Thickness Design Recommendations

Our pavement thickness recommendations for rigid Portland cement concrete pavement are based on an assumed traffic volume, and a modulus of subgrade reaction obtained from the California Bearing Ratio test performed on the subgrade soil sample obtained during our field study. A modulus of subgrade reaction of 90 psi/inch was used in our analysis. The rigid pavement may be designed using a concrete thickness of five and one-half (5½) inches for an estimated 18,000 pounds equivalent single axle load (ESAL) less than 150,000.

Concrete sidewalks should have a nominal thickness of four (4) inches if no vehicle traffic will be allowed on them and at least Five and one-half (5½) inches where vehicle traffic will be on or cross the sidewalks. The concrete sidewalks and aprons may be placed on a leveling course of aggregate base course material. The leveling course should be at least four (4) inches thick and compacted as discussed above for aggregate base course.

The concrete should be supported on prepared subgrade which is at least one (1) foot thick. The prepared subgrade should consist of either compacted structural fill to establish subgrade elevation or natural soils which are scarified to a depth of one (1) foot moisture conditioned to near optimum moisture content and recompacted to at least ninety (90) percent of the maximum dry density as defined by ASTM D1557, modified moisture content-dry density relationship test. If during subgrade preparation any loose or yielding area or any areas of poorly constructed man-placed fill are encountered they should be removed and replaced with compacted structural fill. Suggestions for constructing compacted structural fill are presented below.

Subgrade preparation may result in areas which yield. If yielding areas are encountered during subgrade preparation in the concrete paved areas, such as the apron or sidewalk areas, the subgrade may be overexcavated to a depth of about one foot below the subgrade elevation and backfilled with a compacted structural fill. The structural fill material may aid in construction of the concrete paved areas subgrade. The structural fill material should be an Aggregate subbase course or aggregate base course type material placed and compacted as discussed above.

The Portland cement concrete should be from an approved concrete mix design stating the proportions and mixtures of the mix. We recommend verification of the mix design prior to paving. The coarse and fine aggregate used in the concrete mix should be tested for their suitability for use as concrete aggregate.

The concrete pavement should be appropriately jointed and structurally reinforced to help control the location of cracking. The structural engineer should be contacted to provide structural design recommendations or structural reinforcement and joint design of the concrete pavement.

11.0 BACKFILL

Backfill areas and utility trench backfill should be constructed such that the backfill will not settle after completion of construction, and that the backfill is relatively impervious for the upper few feet. The backfill material should be free of trash and other deleterious material. It should be moisture conditioned and compacted to at least ninety (90) percent relative compaction using a modified moisture content-dry density (Proctor) relationship test (ASTM D1557). Only enough water should be added to the backfill material to allow proper compaction. Do not pond, puddle, float or jet backfill soil materials.

Improperly placed backfill material will allow water migration more easily than properly recompacted fill. Improperly compacted fill is likely to settle, creating a low surface area which further enhances water accumulation and subsequent migration to the foundation soils.

Improperly placed backfill will allow water to migrate along the utility trench or backfill areas to gain access to the subgrade support soils with subsequent mobilization of the swell or settlement mechanism resulting in movement of the supported structure. Moisture migration could also result in the inconvenience of free water in the crawl space.

Backfill placement techniques should not jeopardize the integrity of existing structural members. We recommend recently constructed concrete structural members be appropriately cured prior to adjacent backfilling.

12.0 SURFACE DRAINAGE

The foundation soil materials should be prevented from becoming wetted after construction. Post construction wetting of the soil support soil materials can initiate swell potential or settlement potential as well as decrease the bearing capacity of the support soil materials. Protecting the foundation from wetting can be aided by providing positive and rapid drainage of surface water away from the structure.

The final grade of the ground surface adjacent to the structure should have a well defined slope away from the foundation walls on all sides. The ability to establish proper site surface drainage away from the structure foundation system may be influenced by the existing topography, existing structure elevations and the grades and elevations of the ground surface adjacent to the proposed structure. We suggest where possible a minimum fall of the surface grade away from the structure be that which will accommodate other project grading constraints and provide rapid drainage of surface water away from the structure. If there are no other project constraints we suggest a fall of about one (1) foot in the first ten (10) feet away from the structure foundation. Appropriate surface drainage should be maintained for the life of the project. Future landscaping plans should include care and attention to the potential influence on the long term performance of the foundation and/or crawl space if improper surface drainage is not maintained.

Downspouts and faucets should discharge onto splash blocks that extend beyond the limits of the backfill areas. Splash blocks should be sloped away from the foundation walls. Snow storage

areas should not be located next to the structure. Proper surface
drainage should be maintained from the onset of construction
through the proposed project life.

If significant water concentration and velocity occurs erosion
may occur. Erosion protection may be considered to reduce soil
erosion potential. A landscape specialist or civil engineer should
be consulted for surface drainage design, erosion protection and
landscaping considerations.

13.0 LANDSCAPE IRRIGATION

An irrigation system should not be installed next to foundations,
concrete flatwork or paved areas. If an irrigation system is in-
stalled, the system should be placed so that the irrigation water
does not fall or flow near foundations, flatwork or pavements. The
amount of irrigation water should be controlled.

We recommend that wherever possible xeriscaping concepts be used.
Generally, the xeriscape includes planning and design concepts
which will reduce irrigation water. The reason we suggest xeri-
scape concepts for landscaping is because the reduced landscape
water will decrease the potential for water to influence the long
term performance of the structure foundations and flatwork. Many
publications are available which discuss xeriscape. Colorado State
University Cooperative Extension has several useful publications
and most landscape architects are familiar with the subject.

Due to the expansive nature of the soils tested we suggest that
the owner consider landscaping with only native vegetation which
requires only natural precipitation to survive. Additional irriga-
tion water will greatly increase the likelihood of damage to the
structure as a result of volume changes of the material supporting
the structure.

14.0 SOIL CORROSIVITY TO CONCRETE

Chemical tests were performed on a sample of soil obtained during
the field study. The soil sample was tested for pH and water
soluble sulfates. The results are presented in Appendix B. The
test results indicate a water soluble sulfate content greater than
160 parts per million. Based on the American Concrete Institute
(ACI) information, a water soluble sulfate content greater than 160
parts per million indicates moderate to severe exposure to sulfate
attack on concrete. We suggest sulfate resistant cement be used in
concrete which will be in contact with the on site soils. American
Concrete Institute recommendations for sulfate resistant cement

based on the water soluble sulfate content should be used. The American Concrete Institute recommends a maximum water/cement ratio of 0.5 for concrete where moderate exposure to sulfate attack will occur and a maximum water/cement ratio of 0.45 for concrete where severe exposure to sulfate attack will occur.

15.0 RADON CONSIDERATIONS

Our experience indicates that many of the soils in western Colorado produce small quantities of radon gas. Radon gas may tend to collect in closed poorly ventilated structures. Radon considerations are presented in Appendix C.

16.0 POST DESIGN CONSIDERATIONS

The project geotechnical engineer should be consulted during construction of the project to observe site conditions and open excavations during construction and to provide materials testing of soil and concrete.

This subsurface soil and foundation condition study is based on limited sampling; therefore, it is necessary to assume that the subsurface conditions do not vary greatly from those encountered in the field study. Our experience has shown that significant variations are likely to exist and can become apparent only during additional on site excavation. For this reason, and because of our familiarity with the project, Lambert and Associates should be retained to observe foundation excavations prior to foundation construction, to observe the geotechnical engineering aspects of the construction and to be available in the event any unusual or unexpected conditions are encountered. The cost of the geotechnical engineering observations and material testing during construction or additional engineering consultation is not included in the fee for this report. We recommend that your construction budget include site visits early during construction schedule for the project geotechnical engineer to observe foundation excavations and for additional site visits to test compacted soil.

We recommend that the observation and material testing services during construction be retained by the owner or the owner's engineer or architect, not the contractor, to maintain third party credibility. We are experienced and available to provide material testing services. We have included a copy of a report prepared by Van Gilder Insurance which discusses testing services during construction. It is our opinion that the owner, architect and engineer be familiar with the information. If you have any questions regarding this concept please contact us.

We suggest that your construction plans and schedule include provisions for geotechnical engineering observations and material testing during construction and your budget reflect these provisions.

It is difficult to predict if unexpected subsurface conditions will be encountered during construction. Since such conditions may be found, we suggest that the owner and the contractor make provisions in their budget and construction schedule to accommodate unexpected subsurface conditions.

16.1 Structural Fill Quality

It is our understanding that the proposed development may include compacted structural fill. The quality of compacted structural fill will depend on the type of material used as structural fill, fill lift thickness, fill moisture condition and compactive effort used during construction of the structural fill. Engineering observation and testing of structural fill is essential as an aid to safeguard the quality and performance of the structural fill.

Testing of the structural fill normally includes tests to determine the grain size distribution, swell potential and moisture-density relationship of the fill material to verify the material suitability for use as structural fill. As the material is placed the in-place moisture content and dry density are tested to indicate the relative compaction of the placed structural fill. We recommend that your budget include provisions for observation and testing of structural fill during construction.

Testing of the compacted fill material should include tests of the moisture content and density of the fill material placed and compacted prior to placement of additional fill material. We suggest that a reasonable number of density tests of the fill material can best be determined on a site, material and construction basis although as a guideline we suggest one test per about each 300 to 500 square feet of each lift of fill material. Utility trench backfill may need to be tested about every 100 linear feet of lift of backfill.

16.2 Concrete Quality

It is our understanding current plans include reinforced structural concrete for foundations and walls and may include concrete slabs on grade and pavement. To insure concrete members perform as intended, the structural engineer should be consulted and should address factors such as design loadings, anticipated movement and deformations.

The quality of concrete is influenced by proportioning of the concrete mix, placement, consolidation and curing. Desirable qualities of concrete include compressive strength, water tightness and resistance to weathering. Engineering observations and testing of concrete during construction is essential as an aid to safeguard the quality of the completed concrete.

Testing of the concrete is normally performed to determine compressive strength, entrained air content, slump and temperature. We recommend that your budget include provisions for testing of concrete during construction. We suggest that a reasonable frequency of concrete tests can best be determined on a site, materials and construction specific basis although as a guideline American Concrete Institute, ACI, suggests one test per about each fifty (50) cubic yards or portion thereof per day of concrete material placed.

17.0 LIMITATIONS

It is the owner's and the owner's representatives' responsibility to read this report and become familiar with the recommendations and suggestions presented. We should be contacted if any questions arise concerning the geotechnical engineering aspects of this project as a result of the information presented in this report.

The scope of services for this study does not include either specifically or by implication any environmental or biological (such as mold, fungi, bacteria, etc.) Assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be performed.

The proposed building site contains soils with significant swell potential. For this reason we suggest that, in compliance with Senate Bill 13 you provide a copy of this geotechnical engineering report, a copy of Special Publication 11, "Home Construction on Shrinking and Swelling Soils", and a copy of Special Publication 14, "Home Landscaping and Maintenance on Swelling Soils" to the owner and/or future owners. We are available to discuss this with you.

The recommendations outlined above are based on our understanding of the currently proposed construction. We are available to discuss the details of our recommendations with you and revise them where necessary. This geotechnical engineering report is based on the proposed site development and scope of services as discussed with Mr. Ron Abeloe, on the type of construction planned, existing site conditions at the time of the field study, and on our findings. Should the planned, proposed use of the site be altered, Lambert and Associates must be contacted, since any such changes may make our suggestions and recommendations inappropriate. This report should be used ONLY for the planned development for which this report was tailored and prepared, and ONLY to meet information needs of the owner and the owner's representatives. In the event that any changes in the future design or location of the building are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or verified in writing. It is recommended that the geotechnical engineer be provided the opportunity for a general review of the final project design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications.

This report does not provide earthwork specifications. We can provide guidelines for your use in preparing project specific earthwork specifications. Please contact us if you need these for your project.

This report presents both suggestions and recommendations. The suggestions are presented so that the owner and the owner's representatives may compare the cost to the potential risk or benefit for the suggested procedures.

This report contains suggestions and recommendations which are intended to work in concert with recommendations provided by the other design team members to provide somewhat predictable foundation performance. If any of the recommendations are not included in the design and construction of the project it may result in unpredictable foundation performance or performance different than anticipated. We recommend that we be requested to provide geotechnical engineering observation and materials testing during the construction phase of the project as discussed in this report. The purpose for on site observation and testing by us during construction is to help provide continuity of service from the planning of the project through the construction of the project. This service will also allow us to revise our recommendations if conditions occur or are discovered during construction that were not evidenced during the initial study. We suggest that the owner and the contractor make provisions in their construction budget and construction schedule to accommodate unexpcted subsurface conditions.

We represent that our services were performed within the limits prescribed by you and with the usual thoroughness and competence of the current accepted practice of the geotechnical engineering profession in the area. No warranty or representation either expressed or implied is included or intended in this report or our contract. We are available to discuss our findings with you. If you have any questions please contact us. The supporting data for this report is included in the accompanying figures and appendices.

This report is a product of Lambert and Associates. Excerpts from this report used in other documents may not convey the intent or proper concepts when taken out of context, or they may be misinterpreted or used incorrectly. Reproduction, in part or whole, of this document without prior written consent of Lambert and Associates is prohibited.

This report and information presented can be used only for this site, for this proposed development, and only for the client for whom our work was performed. Any other circumstances are not appropriate applications of this information. Other development plans will require project specific review by us.

We have enclosed a copy of a brief discussion about geotechnical engineering reports published by Association of Soil and Foundation Engineers for your reference.

If you plan to utilize the services of Home Buyers Warranty for the proposed development you should become familiar with their construction criteria prior to beginning your development. For further information we suggest you contact Home Buyers Warranty, 2675 S. Abilene Street, Aurora, Colorado, 80014, 1-800-488-8844 for a copy of their manual.

Please call when further consultation or observations and tests are required.

If you have any questions concerning this report or if we may be of further assistance, please contact us.

Respectfully submitted;

LAMBERT AND ASSOCIATES

Norman W. Johnston

Norman W. Johnston, P.E.
Manager Geotechnical Engineer

Reviewed by:

Dennis D. Lambert, P.E.
Principal Geotechnical Engineer

NWJ/nr

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THE PROFESSIONAL LIABILITY PERSPECTIVE

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WHO HIRES THE TESTING LABORATORY?

It is one of those relatively small details in the overall scheme of things. Independent testing may be required by local building codes, or it may be insisted upon by lenders. Additional testing can usually be ordered by the design team during construction. Whatever the source of the requirement, many owners perceive it to be an unnecessary burden—an additional cost imposed principally for someone else's benefit.

What does this have to do with you? You may be the only one in a position to influence the use of testing and inspection services so they become more, rather than less likely to contribute to a successful outcome. There seems to be an almost irresistible inclination on the part of some owners to cast aside their potential value to the project in favor of the administrative and financial convenience of placing responsibility for their delivery into the hands of the general contractor.

Resist this inclination where you can. It is not in your client's best interests, and it is certainly not in yours. There are important issues of quality and even more important issues of life safety at stake. In the complex environment of today's construction arena, it makes very little sense for either of you to give up your control of quality control. Yet it happens altogether too often.

What's Behind this Misadventure?

The culprit seems to be the Federal Government. In the 1960's, someone came up with the idea that millions could be saved by eliminating the jobs of Federal workers engaged in construction inspection. The procurement model used to support this stroke of genius was the manufacturing segment of the economy, where producers of goods purchased by the Government had been required for years to conduct their own quality assurance programs. The result was a trendy new concept in Federal construction known as Contractor Quality Control (CQC).

It was a dumb idea. Costs were simply shifted from the Federal payroll to capital improvement budgets. Government contractors, selected on the basis of the lowest bid, were handed resources to assure the quality of their own performance. Some did so; many did not. All found themselves caught up in an impossible conflict between the demands of time and cost, on one hand, and the dictates of quality, on the other.

CQC was opposed by the Associated General Contractors of America, by independent testing laboratories, by the design professions, and by those charged with front-line responsibility for quality control in the Federal Agencies. Eventually, even the General Accounting Office came to the conclusion that it ought to be abandoned. But, once set in motion and fueled by the pervasive influence of the Federal Government, the idea spread—first to state and local governments; finally, to the private sector.

Why would the private sector embrace such an ill-conceived notion? Because so many owners view testing and inspection as an undertaking which simply duplicates something they are entitled to in any event. They are confident they will be protected by contract documents which cover every detail and contingency. They look to local building inspectors to assure compliance with codes. And they fully expect the design team to fulfill its obligation to safeguard the quality of the work.

A Fox in the Henhouse

If testing is perceived as little more than an unnecessary, but unavoidable expense, why not make the general contractor responsible for controlling the cost? It may produce a savings, and it certainly eliminates an adminstrative headache. If contractual obligations dealing with the project schedule and budget can be enforced, surely those governing quality can be enforced, as well. Possibly so, but who is going to do it?

Some testing consultants will not accept CQC work. The reasons they give come from firsthand experience. They include: 1) inadequate to barely adequate scope, 2) selection based on the lowest bid; 3) non-negotiable contract terms inappropriate to the delivery of a professional service; 4) intimidation of inspectors by field supervisors; and 5) suppression of low or failing test results. This ought to be fair warning to any owner.

Keeping Both Hands on the Wheel

The largest part of the problem, from your point of view, is one of artful persuasion. If you cannot convince your client of the value of independent testing and inspection, no one can. Yet, if you do not, you are likely to find yourself responsible for an assurance of quality you are in no position to deliver. How can you keep quality control where it belongs and, in the process, prevent the owner from compromising his or her interests in the project as well as yours? Consider these suggestions:

1. Put the issue on an early agenda. It needs your attention. Anticipate the owner's inclination to avoid dealing with testing and inspection, and explain its importance to the success of the project. Persist, if you can, until your client agrees to hire the testing laboratory independently and to establish an adequate budget to meet the anticipated costs. A testing consultant hired by the owner cannot be fired by the general contractor for producing less than favorable results.

2. Tailor the testing requirements carefully. Scissors and paste can be your very worst enemies. Specify what the job requires, retain control of selection and hiring, make certain the contractor's responsibilities for notification for scheduling purposes are clear, and require that copies of all reports be distributed by the laboratory directly to you.

3. Insist on a preconstruction testing conference. It can be an essential element of effective coordination. Include the owner, the general contractor, major subcontractors, the testing consultant, and the design team. Review your requirements, the procedures to be followed, and the responsibilities of each of the parties. Have the testing consultant prepare a conference memorandum for distribution to all participants.

4. Monitor tests and inspections closely. Make certain your field representative is present during tests and inspections, so that deficiencies in procedures or results can be reported and acted upon quickly. Scale back testing if it becomes clear it is appropiate to do so under the circumstances; do not hesitate to order additional tests if they are required.

5. Finally, keep your client informed. Without your help, he or she is not likely to understand what the test results mean, nor will your actions in response to them make much sense. If additional testing is called for, explain why. Remember, it is an unexpected and, possibly, unbudgeted additional cost for which you will need to pave the way. In this sense, independent testing and inspection can serve an important, secondary purpose. You might view it as a communications resource. Use it in this way, and it just may yield unexpected dividends.

IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, due in large measure to programs and publications of ASFE/ The Association of Engineering Firms Practicing in the Geosciences.

The following suggestions and observations are offered to help you reduce the geotechnical-related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include: the general nature of the structure involved, its size and configuration; the location of the structure on the site and its orientation; physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program. To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of the report may affect its recommendations.

Unless your consulting geotechnical engineer indicates otherwise, *your geotechnical engineering report should not be used:*

- When the nature of the proposed structure is changed, for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size or configuration of the proposed structure is altered,
- when the location or orientation of the proposed structure is modified,
- when there is a change of ownership, or
- for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems which may develop if they are not consulted after factors considered in their report's development have changed.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing are extrapolated by geotechnical engineers who then render an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those inferred to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. The actual interface between materials may be far more gradual or abrupt than a report indicates. Actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact.* For this reason, *most experienced owners retain their geotechnical consultants through the construction stage,* to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly-changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time.* Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

Geotechnical engineers' reports are prepared to meet the specific needs of specific individuals. A report prepared for a consulting civil engineer may not be adequate for a construction contractor, or even some other consulting civil engineer. Unless indicated otherwise, this report was prepared expressly for the client involved and expressly for purposes indicated by the client. Use by any other persons for any purpose, or by the client for a different purpose, may result in problems. *No individual other than the client should apply this report for its intended purpose without first conferring with the geotechnical engineer. No person should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.*

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid these problems, the geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy of their plans and specifications relative to geotechnical issues.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE ENGINEERING REPORT

Final boring logs are developed by geotechnical engineers based upon their interpretation of field logs (assembled by site personnel) and laboratory evaluation of field samples. Only final boring logs customarily are included in geotechnical engineering reports. *These logs should not under any circumstances be redrawn* for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes and unanticipated costs are the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, *give contractors ready access to the complete geotechnical engineering report* prepared or authorized for their use. Those who do not provide such access may proceed under the *mistaken* impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes which aggravate them to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical consultants. To help prevent this problem, geotechnical engineers have developed model clauses for use in written transmittals. These are *not* exculpatory clauses designed to foist geotechnical engineers' liabilities onto someone else. Rather, they are definitive clauses which identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report, and you are encouraged to read them closely. Your geotechnical engineer will be pleased to give full and frank answers to your questions.

OTHER STEPS YOU CAN TAKE TO REDUCE RISK

Your consulting geotechnical engineer will be pleased to discuss other techniques which can be employed to mitigate risk. In addition, ASFE has developed a variety of materials which may be beneficial. Contact ASFE for a complimentary copy of its publications directory.

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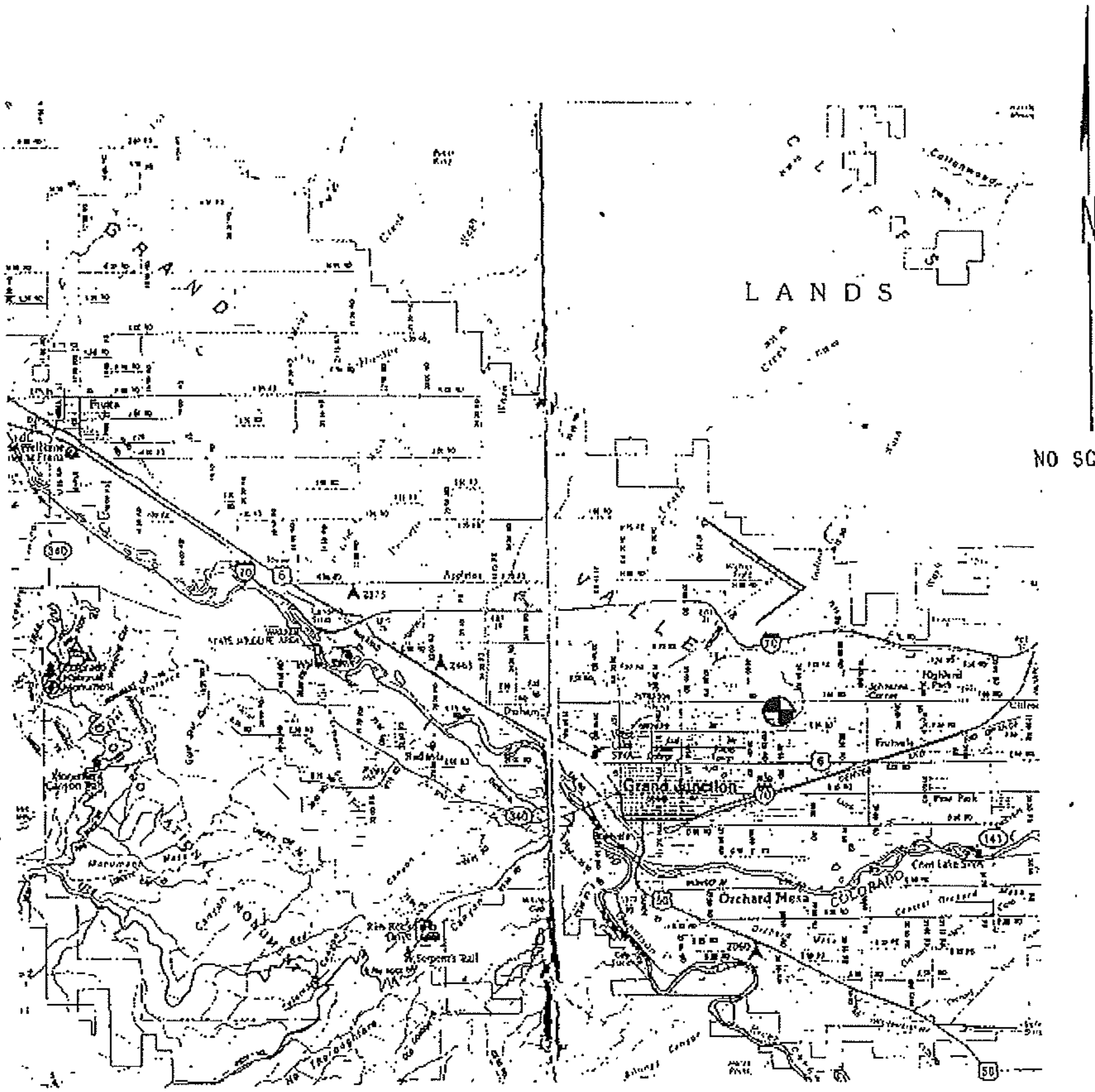

NO SCALE

Indicates approximate project location

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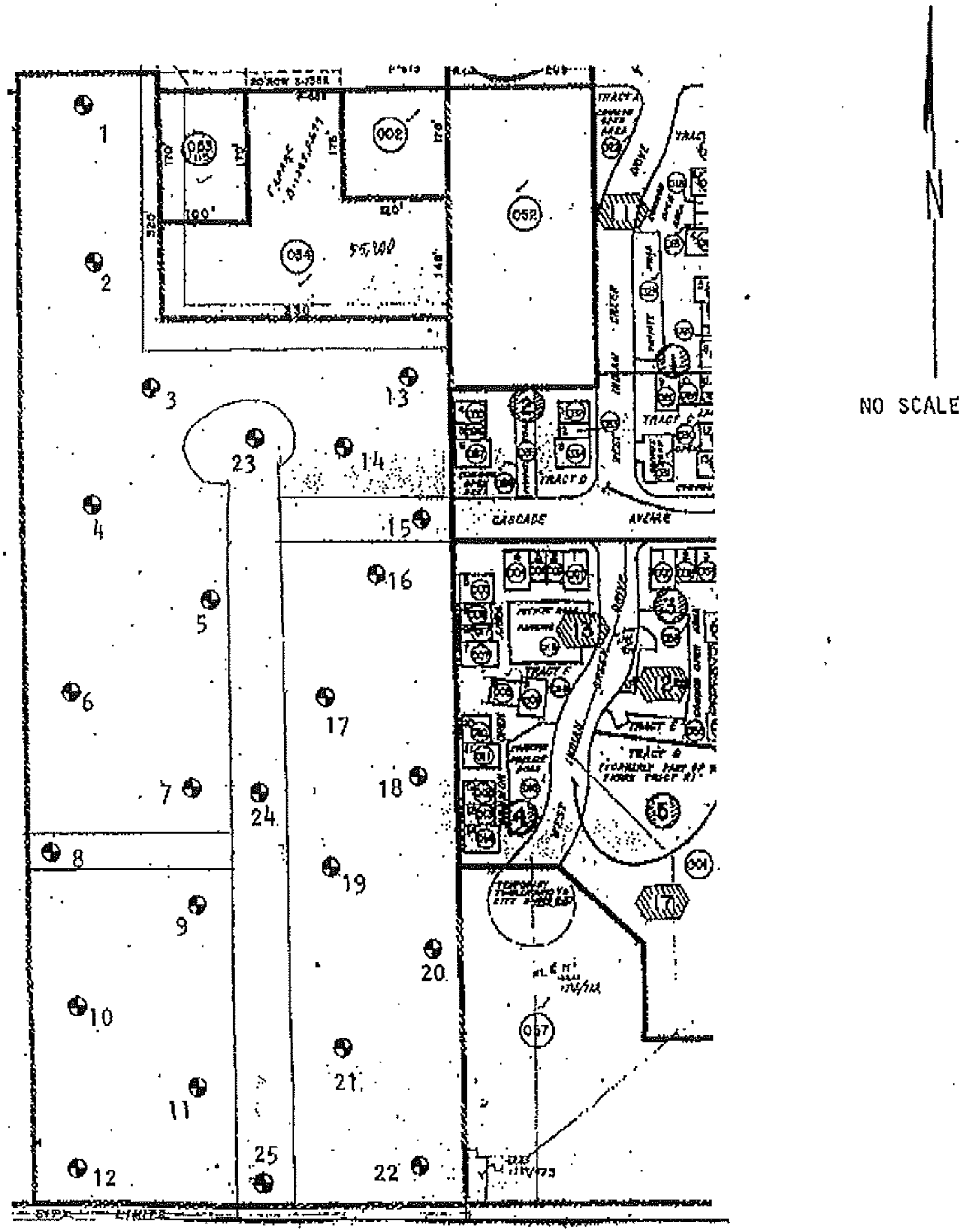
PROJECT VICINITY MAP

Lambert and Associates	**Project No.:** M03032GE
	Date: 3/13/03
	Figure: 1



25 ⊕ Indicates approximate test boring locations

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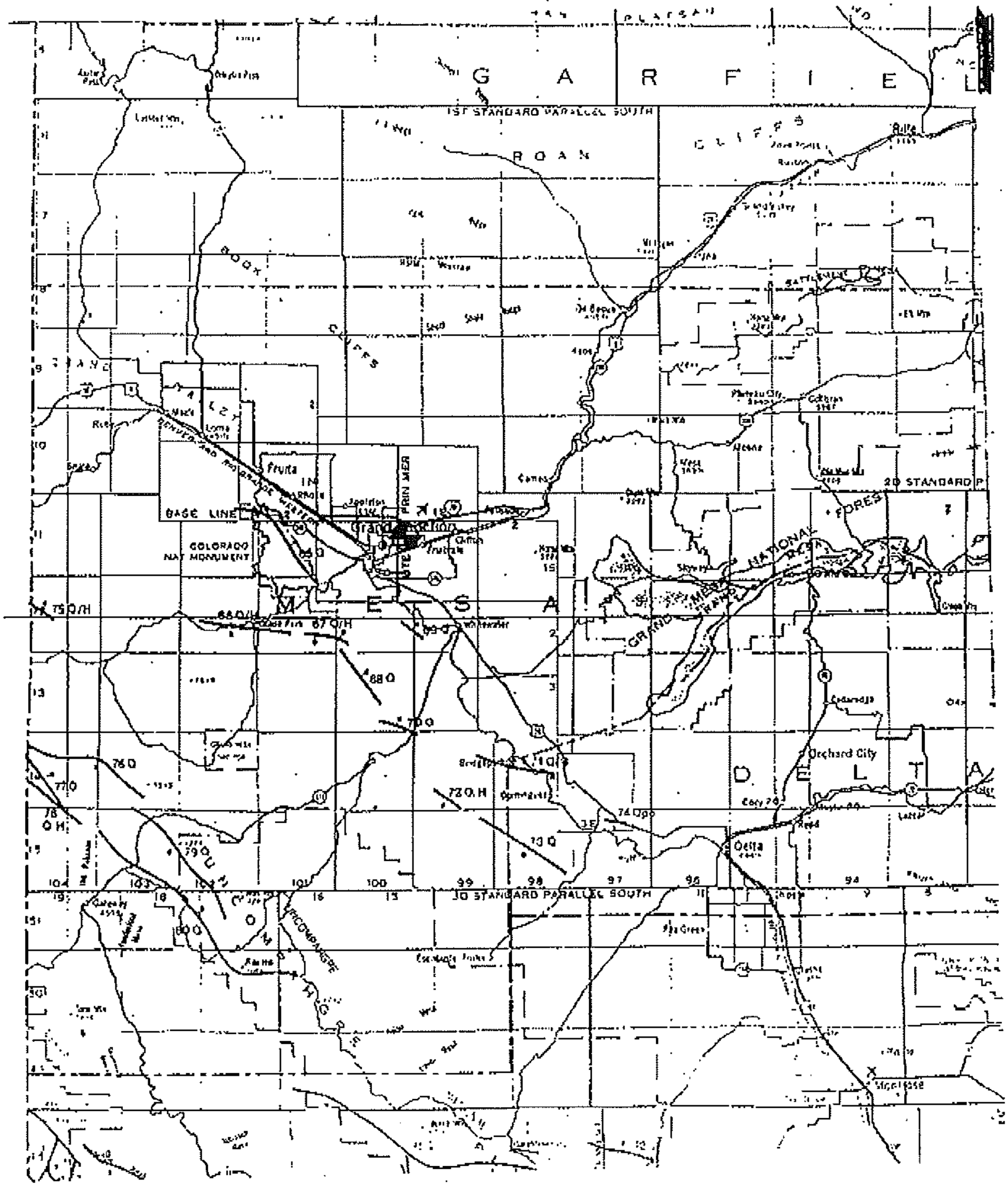
TEST BORING LOCATION SKETCH

Lambert and Associates	Project No.:	M030320E
	Date:	3/13/03
	Figure:	2

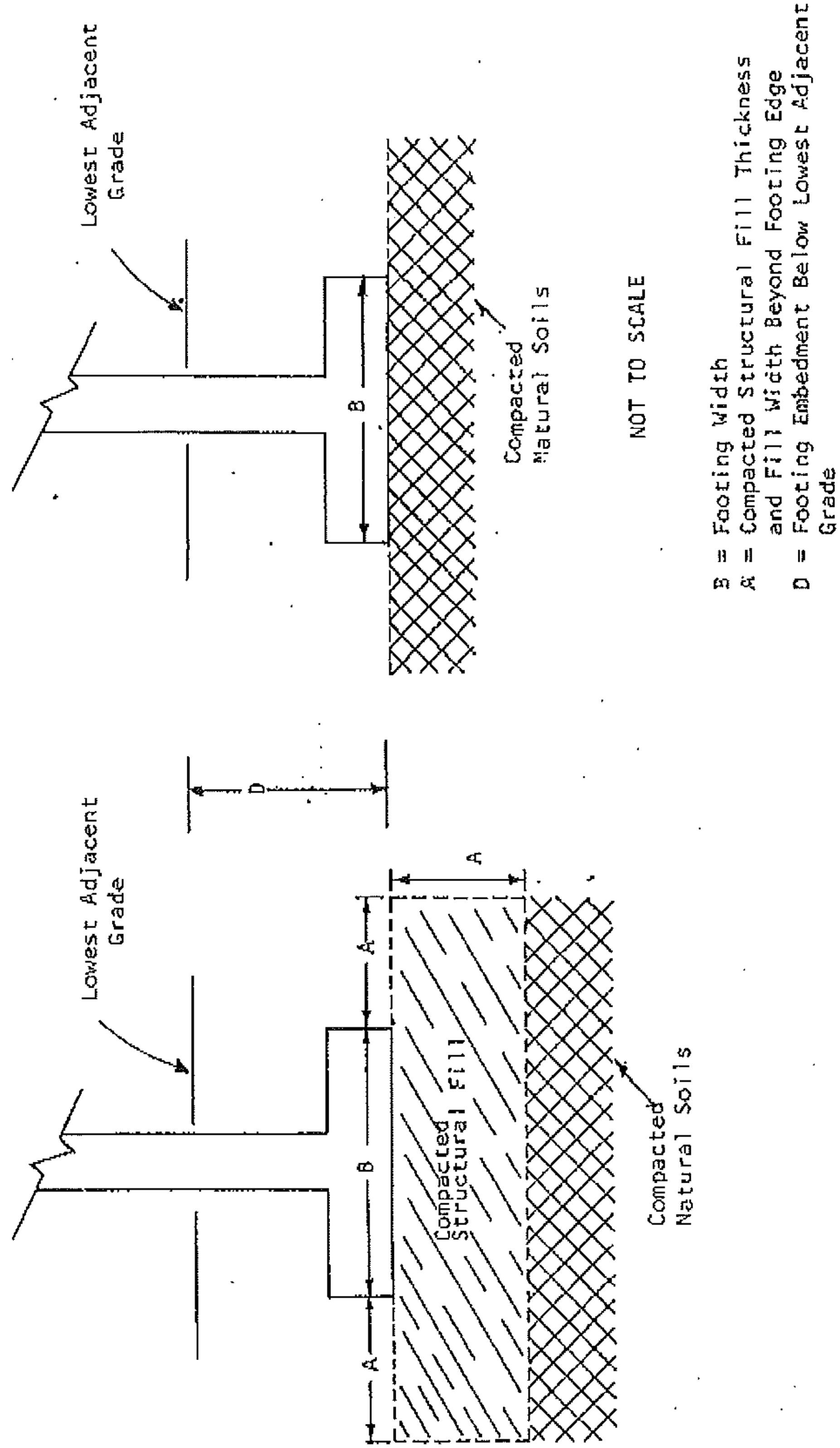


Indicates approximate project location

FAULT LOCATION SKETCH

Lambert and Associates

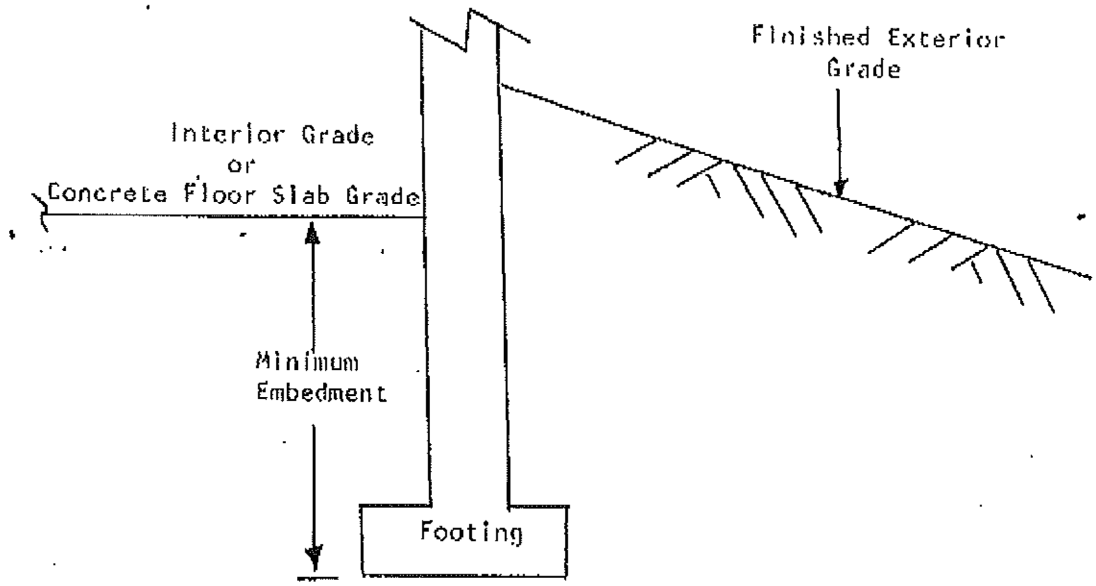
Project No.	M03032GE
Date:	3/13/03
Figure:	3

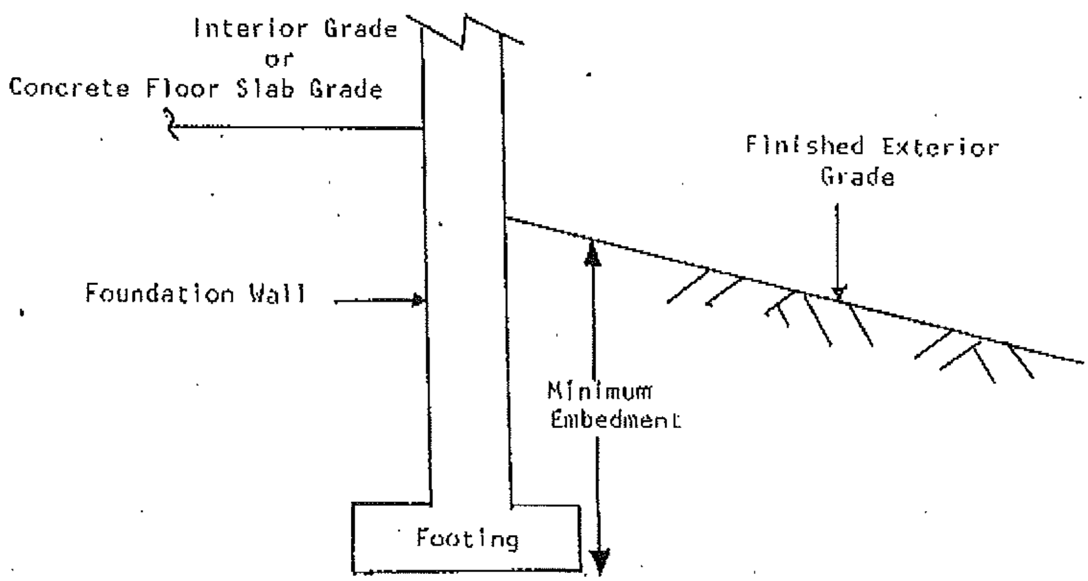


CONCEPTUAL SKETCH OF FOOTING SUBGRADE TREATMENT

Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	4





EMBEDMENT CONCEPT

NO SCALE

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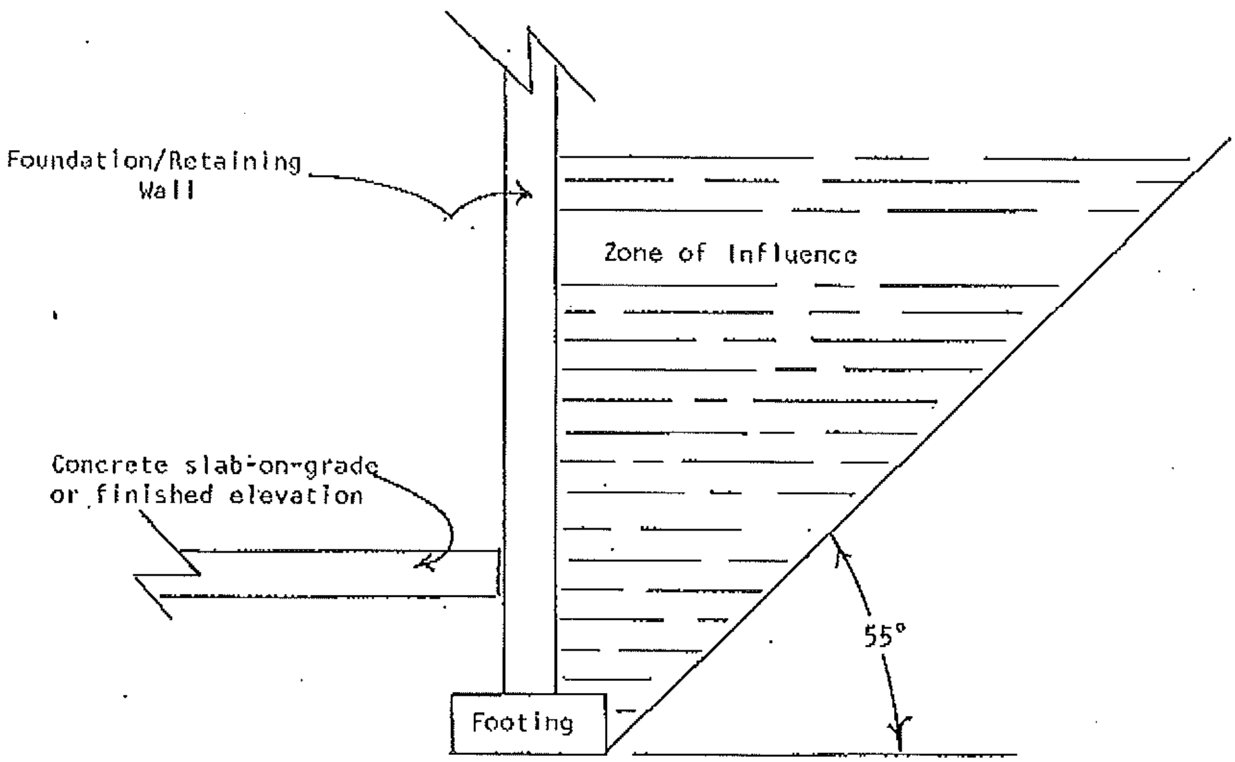
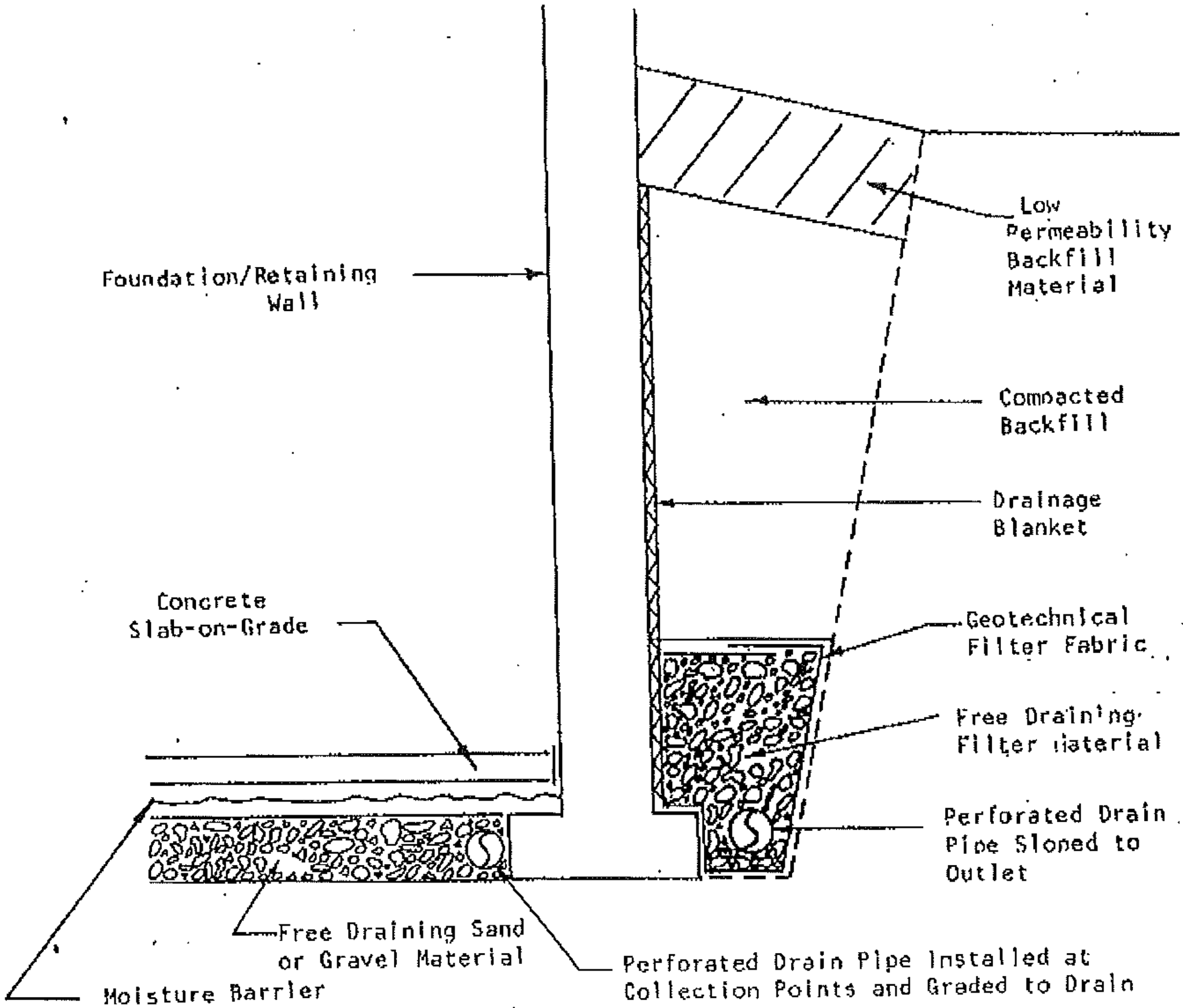
Project No.:	M03032GE
Date:	3/13/03
Figure:	5



BACKFILL ZONE OF INFLUENCE CONCEPT

Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	6



This sketch is to show concept only. The text of our report should be consulted for additional information.

CONCEPTUAL SKETCH OF FOUNDATION DRAIN SYSTEM

Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	7

APPENDIX A

The field study was performed on January 30, 2003. The field study consisted of logging and sampling the soils encountered in twenty-five (25) auger advanced test borings. The approximate locations of the test borings are shown on Figure 2. The log of the soils encountered in the test borings are presented on Figures A2 through A26.

The test borings were logged by Lambert and Associates and samples of significant soil types were obtained. The samples were obtained from the test borings using a Modified California Barrel sampler and bulk disturbed samples were obtained. Penetration blow counts were determined using a 140 pound hammer free falling 30 inches. The blow counts are presented on the logs of the test borings such as 8/6 where 8 blows with the hammer were required to drive the sampler 6 inches.

The engineering field description and major soil classification are based on our interpretation of the materials encountered and are prepared according to the Unified Soil Classification System, ASTM D2488. The description and classification which appear on the test boring log is intended to be that which most accurately describes a given interval of the test boring (frequently an interval of several feet). Occasionally discrepancies occur in the Unified Soil Classification System nomenclature between an interval of the soil log and a particular sample in the interval. For example, an interval on the test boring log may be identified as a silty sand (SM) while one sample taken within the interval may have individually been identified as a sandy silt (ML). This discrepancy is frequently allowed to remain to emphasize the occurrence of local textural variations in the interval.

The stratification lines presented on the logs are intended to present our interpretation of the subsurface conditions encountered in the test borings. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

KEY TO
LOG OF TEST BORING

Date Drilled ____________ Field Engineer ____________ Boring Number ____________

Location ______________________________ Elevation ____________

Diameter ____________ Total Depth ____________

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
				Sand, silty, medium dense, moist, tan, (SM) Unified Soil Classification	Notes in this column indicate tests performed and test results if not plotted.
				Indicates Bulk Bag Sample	DD: Indicates dry density in pounds per cubic foot
	5	C		Indicates Drive Sample Indicates Sampler Type: C - Modified California St - Standard Split Spoon H - Hand Sampler	MC: Indicates moisture content as percent of dry unit weight LL: Indicates Liquid Limit
			7/12	Indicates seven blows required to drive the sampler twelve inches with a hammer that weighs one hundred forty pounds and is dropped thirty inches.	PL: Indicates Plastic Limit PI: Indicates Plasticity Index
	10			BOUNCE: Indicates no further penetration occurred with additional blows with the hammer	
				NR: Indicates no sample recovered	
				CAVED: Indicates depth the test boring caved after drilling	
▽	15			Indicates the location of free subsurface water when measured	
				CLAY — NOTE: Symbols are often used only to help visually identify the described information presented on the log.	
				SILT	
	20			SAND	
				GRAVEL	
				CLAYSTONE	
				SANDSTONE	
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A1

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Johnston Boring Number 1

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		Bulk		Clay,silty,sandy,gravelly,stiff, mediu moist,brown (CL)	
	5	C	8/6 7/6		Swell-Consolidation Test: MC: 9.7% DD: 90.0 pcf
	10	C	50/6	Formational material,silty clay shale hard,gray,Mancos Shale formation	
				Harder with depth	
	15			Bottom of test boring 1 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A2

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Johnston **Boring Number** 2

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
	0–3	Bulk		Clay, silty, sandy, medium stiff, moist, brown (CL) man placed fill	
				Clay, silty, stiff, medium moist, brown (CL)	
	5	C	12/6 21/6	Formational material, silty clay shale, hard, gray, Mancos Shale formation	Swell-Consolidation Test: MC: 10.6% DD: 108.0 pcf
	10				
	15			Bottom of test boring 2 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development **Project Number** M03032GE **Figure** A3

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Johnston Boring Number 3

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
		Bulk		Clay,silty,stiff,medium moist,brown (CL)	
				Formational material,silty clay shale hard,gray,Mancos Shale formation	
		C	50/5	NR	
	5				
				Harder with depth	
		C	50/2	NR	
	10				
	15			Bottom of test boring 3 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A4

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Kintz Boring Number 4

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 14 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
	0–4	Bulk		Clay, silty, sandy, gravelly, few cobbles; medium stiff, slightly moist, brown-tan (CL) man placed fill	
	5	C	44/6 50/5	Formational material, silty clay shale, hard to very hard, brown-gray, Mancos Shale formation	
	10	C	24/6 25/6	Color change with depth	
	15			Bottom of test boring 4 at 14 feet	
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A5

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Johnston Boring Number 5

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		Bulk		Clay, silty, sandy, gravelly, medium stiff, moist, brown (CL) man placed fill	
				Clay, silty, stiff, medium moist, brown (CL)	
	5	C	34/6 42/6	Formational material, silty clay shale hard, gray, Mancos Shale formation	Swell-Consolidation Test: MC: 12.3% DD: 117.0 pcf Direct Shear Strength Test: MC: 11.6% DD: 123.0 pcf
	10	C	50/2 NR		
	15			Bottom of test boring 5 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A6

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Kintz **Boring Number** 6

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 14 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		Bulk		Clay,silty,sandy,few gravel,medium stiff,slighly moist,brown-tan(CL) man placed fill	
				Clay,silty,sandy,medium stiff,slightly moist,brown-tan (CL)	
				Formational material,silty clay shale, stiff to hard,brown-gray Mancos Shale formation	
	5	C	26/6 40/6		Swell-Consolidation Test: MC: 13.1% DD: 116.0 pcf Direct Shear Strength Test: MC: 11.5% DD: 114.0 pcf
	10	C	35/6 35/6	Harder with depth	
	15			Bottom of test boring 6 at 14 feet	
	20				
	25				

Project Name Parkerson Property Development **Project Number** M03032GE **Figure** A7

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Kintz Boring Number 7

Location See test boring location sketch Elevation

Diameter 4 Inches Total Depth 14 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
				Clay, silty, stiff, slightly moist, brown-tan (CL)	
		Bulk		Formational material, silty clay shale, stiff to hard, brown, Mancos Shale formation	
	5	C	50/5		
	10				
				Bottom of test boring 7 at 14 feet	
	15				
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A8

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Kintz Boring Number 8

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 5 feet Depth to Water at Time of Drilling None

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0–5	Bulk		Clay, slightly silty, sandy, gravelly, medium dense, slightly moist, brown-gray (CL-SP)	
	5			Bottom of test boring 8 at 5 feet	
	10				
	15				
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A9

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Kintz Boring Number 9

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 19 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		Bulk		Clay, silty, slightly sandy, soft, slightly moist, tan (CL)	
				More sand at 4 to 6 feet	
	5	C	8/6 11/6		Swell-Consolidation Test: MC: 7.0% DD: 95.0 pcf
				Possible weathered shale at 6 to 9 feet	
	10			Formational material, silty clay shale stiff, brown-tan, Mancos Shale formation	
	15				
	20			Bottom of test boring 9 at 19 feet	
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A10

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Kintz **Boring Number** 10

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 19 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
	0–3	Bulk		Clay,silty,sandy,gravelly,few cobbles dense,slightly moist,brown-tan (CL) man placed fill	
	5	C	18/6 18/6	Formational material,silty clay shale stiff to very stiff,brown,Mancos Shale formation	Direct Shear Strength Test: MC: 11.8% DD: 112.0 pcf
				Very hard with depth	
	10	C	50/5		
	15				
	20			Bottom of test boring 10 at 19 feet	
	25				

Project Name Parkerson Property Development **Project Number** N03032GE **Figure** A11

Lambert and Associates

CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 *Field Engineer* Kintz *Boring Number* 11

Location See test boring location sketch *Elevation*

Diameter 4 inches *Total Depth* 14 feet .Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
	0–4	Bulk		Clay, silty, sandy, medium dense, slightly moist, tan (CL)	
	4–5	C	25/6 40/6		
	5			Formational material, silty clay shale, stiff to very stiff, brown, Mancos Shale formation	
	10				
	14			Bottom of test boring 11 at 14 feet	
	15				
	20				
	25				

Project Name Parkerson Property Development *Project Number* M03032GE *Figure* A12

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LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Kintz **Boring Number** 12

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 14 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
	0–5	Bulk		Sand, gravel, cobbles, clayey, dense, slightly moist, brown-gray-tan (SC-GC) man placed fill	
	5	C	7/6 20/6		Swell-Consolidation Test: MC: 7.2% DD: 87.0 pcf
	10	C	24/6 32/6	Formational material, silty clay shale stiff, brown-brown gray, weathered, Mancos Shale Formation	
	15			Bottom of test boring 12 at 14 feet	
	20				
	25				

Project Name Parkerson Property Development **Project Number** M03032GE **Figure** A13

LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Johnston **Boring Number** 13

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
	0–4	Bulk		Clay, silty, stiff, medium moist, brown (CL) slightly organic to 1 foot	
	5	C	40/6 50/4	Formational material, silty clay shale, hard, gray, Mancos Shale formation	Swell-Consolidation Test: MC: 10.9% DD: 106.0 pcf Direct Shear Strength Test: MC: 11.4% DD: 100.0 pcf
	10	C	50/2	NR	
	15			Bottom of test boring 13 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development **Project Number** M03032GE **Figure** A14

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LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Johnston **Boring Number** 14

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
		Bulk		Clay, silty, stiff, medium moist, brown (CL)	
				Formational material, silty clay shale, hard, gray, Mancos Shale formation	
	5	C	39/6 50/5		
	10				
	15			Bottom of test boring 14 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development **Project Number** M03032GE **Figure** A15

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LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Johnston Boring Number 15

Location See test boring location sketch Elevation

Diameter 8 inches Total Depth 5 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		Bulk		Clay, silty, stiff, medium moist, brown (CL) slightly organic to 1 foot	
				Formational material, silty clay shale, hard, gray, Mancos Shale formation	
	5			Bottom of test boring 15 at 5 feet	
	10				
	15				
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A16

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LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Johnston Boring Number 16

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		Bulk		Clay, silty, stiff, medium moist, brown (CL)	
				Formational material, silty clay shale, hard, gray, Mancos Shale formation	
	5	C	33/6 50/4		
	10				
	15			Bottom of test boring 16 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A17

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LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Johnston **Boring Number** 17

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
		Bulk		Clay,silty,stiff,medium moist,brown (CL) slightly organic to 1 foot	
				Formational material,silty clay shale hard,gray,Mancos Shale formation	
	5	C	50/5		Swell-Consolidation Test: MC: 9.7% DD: 103.0 pcf
	10	C	50/0	NR	
	15			Bottom of test boring 17 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development **Project Number** M03032GE **Figure** A18

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LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Johnston **Boring Number** 18

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
				Clay, silty, stiff, medium moist, brown (CL) slightly organic to 1 foot	
		Bulk		Formational material, silty clay shale, hard, gray, Mancos Shale formation	
	5	C	50/6		
	10	C	50/3	NR	
	15			Bottom of test boring 18 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development **Project Number** M03032GE **Figure** A19

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LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Johnston Boring Number 19

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		Bulk		Clay, silty, stiff, medium moist, brown (CL) slightly organic to 1 foot	
				Formational material, silty clay shale hard, gray, Mancos Shale formation	
	5	C	22/6 42/6		Swell-Consolidation Test: MC: 9.9% DD: 109.0 pcf Direct Shear Strength Test: MC: 10.2% DD: 107.0 pcf
	10				
	15			Bottom of test boring 19 at 15 feet	
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A20

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LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Johnston Boring Number 20

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 15 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
	0–5	Bulk; C	19/6 16/6	Clay, silty, stiff, medium moist, brown (CL) slightly organic to 1 foot, possible man placed fill	Swell-Consolidation Test: MC: 12.7% DD: 96.0 pcf Direct Shear Strength Test: MC: 12.1% DD: 93.0 pcf
	6.5–15	C	35/6 25/3	Formational material, silty clay shale hard, gray, Mancos Shale formation	
	15			Bottom of test boring 20 at 15 feet	

Depth scale: 5, 10, 15, 20, 25

Project Name Parkerson Property Development Project Number M03032GE Figure A21

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LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Kintz Boring Number 21

Location See test boring location sketch Elevation

Diameter 4 inches Total Depth 14 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
		Bulk		Clay, silty, slightly sandy, medium stiff, slightly moist, brown-tan (CL) man placed fill	
				Formational material, silty clay shale, hard to very hard, brown-brown gray Mancos Shale formation	
	5	C	29/6 36/6		
	10				
	15			Bottom of test boring 21 at 14 feet	
	20				
	25				

Project Name Parkerson Property Development Project Number M030320E Figure A22

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CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

LOG OF TEST BORING

Date Drilled 1/30/03 **Field Engineer** Kintz **Boring Number** 22

Location See test boring location sketch **Elevation**

Diameter 4 inches **Total Depth** 14 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	N	Soil Description	Laboratory Test Results
				Clay,silty,slightly sandy,medium stiff,slightly moist,brown-tan (CL)	
		Bulk		Formational material,silty clay shale, hard,brown-browngray,Mancos Shale formation	
	5	C	20/6 34/6		Swell-Consolidation Test: MC: 8.4% DD: .0 pcf
	10	C	44/6 50/5		
	15			Bottom of test boring 22 at 14 feet	
	20				
	25				

Project Name Parkerson Property Development **Project Number** M03032GE **Figure** A23

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Johnston Boring Number 23

Location See test boring location sketch Elevation

Diameter 8 inches Total Depth 5 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0–5	Bulk		Clay,sandy;gravelly,medium stiff, moist,brown (CL) man placed fill	
	5			Bottom of test boring 23 at 5 feet	
	10				
	15				
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A24

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LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Kintz Boring Number 24

Location See test boring location sketch ; Elevation

Diameter 8 inches Total Depth 5 feet Depth to Water at Time of Drilling None encountered.

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
		Bulk		Clay, silty, slightly sandy, soft, slightly moist, brown-gray-tan (CL)	
				Formational material, silty clay shale, stiff to very stiff, brown-brown gray Mancos Shale formation	
	5			Bottom of test boring 24 at 5 feet	
	10				
	15				
	20				
	25				

Project Name Parkerson Property Development Project Number M03032GE Figure A25

LOG OF TEST BORING

Date Drilled 1/30/03 Field Engineer Kintz Boring Number 25

Location See test boring location sketch Elevation

Diameter 8 inches Total Depth 5 feet Depth to Water at Time of Drilling None encountered

Symbol	Depth	Sample Type	Sample N	Soil Description	Laboratory Test Results
	0–5	Bulk		Clay,sand,silty,soft,slightly moist, brown-tan (CL-SP)	
	5			Bottom of test boring 25 at 5 feet	
	10				
	15				
	20				
	25				

Project Name Parkerson Property Development Project Number M030320E Figure A26

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CONSULTING GEOTECHNICAL ENGINEERS AND MATERIAL TESTING

APPENDIX B

The laboratory study consisted of performing:

- Moisture content and dry density tests,
- Swell-consolidation tests,
- Direct Shear Strength tests,
- California bearing ratio tests,
- Moisture Content-dry density relationship tests and
- Chemical tests.

It should be noted that samples obtained using a drive type sleeve sampler may experience some disturbance during the sampling operations. The test results obtained using these samples are used only as indicators of the in situ soil characteristics.

TESTING

Moisture Content and Dry Density

Moisture content and dry density were determined for each sample tested of the samples obtained. The moisture content was determined according to ASTM Test Method D2216 by obtaining the moisture sample from the drive sleeve. The dry density of the sample was determined by using the wet weight of the entire sample tested. The results of the moisture and dry density determinations are presented on the logs of test borings, Figures A2 through A26.

Swell Tests

Loaded swell tests were performed on drive samples obtained during the field study. These tests are performed in general accordance with ASTM Test Method D2435 to the extent that the same equipment and sample dimensions used for consolidation testing are used for the determination of expansion. A sample is subjected to static surcharge, water is introduced to produce saturation, and volume change is measured as in ASTM Test Method D2435. Results are reported as percent change in sample height.

Consolidation Tests

One dimensional consolidation properties of drive samples were evaluated according to the provisions of ASTM Test Method D2435. Water was added in all cases during the test. Exclusive of special readings during consolidation rate tests, readings during an increment of load were taken regularly until the change in sample height was less than 0.001 inch over a two hour period.

The results of the swell-consolidation load test are summarized on Figures B1 through B11, swell-consolidation tests.

It should be noted that the graphic presentation of consolidation data is a presentation of volume change with change in axial load. As a result, both expansion and consolidation can be illustrated.

Direct Shear Strength Tests

Direct shear strength properties of sleeve samples were evaluated in general accordance with testing procedures defined by ASTM Test Method D3080. The direct shear strength test results are tabulated below.

Test Boring	Depth (FT)	Dry Density (PCF)	Moisture Content (Percent)	Cohesion (PSF)	Internal Angle of Friction (Degrees)
5	4	123	11.6	230	26
6	4	114	11.5	610	26
10	4	112	11.8	360	27
13	4	100	11.4	150	26
19	4	107	10.2	275	18
22	4	93	12.1	230	22

California Bearing Ratio Tests

California bearing ratio tests were conducted on select soil samples obtained during our field study. The California bearing ratio tests were conducted in accordance with ASTM Test Method D1883. The results of the California bearing ratio tests are presented on Figure B12.

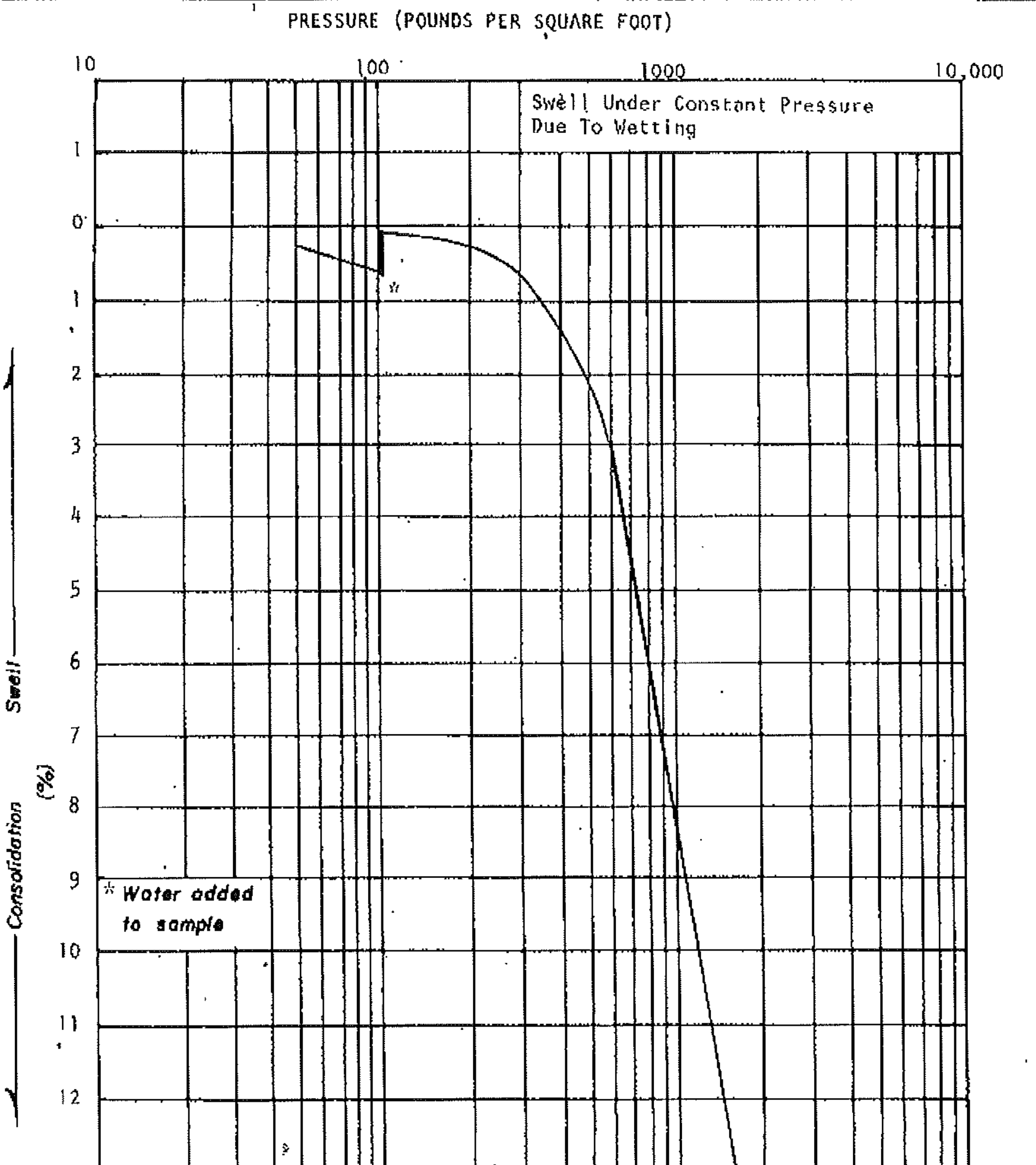
Moisture Content-Dry Density Relationship Tests

Moisture content-dry density relationship tests were conducted on select soils subgrade samples obtained during our field study. The moisture-density relationship tests were conducted in accordance with ASTM Test Method D1557. The results of the moisture-density relationship tests are presented on Figure B13.

Chemical Tests

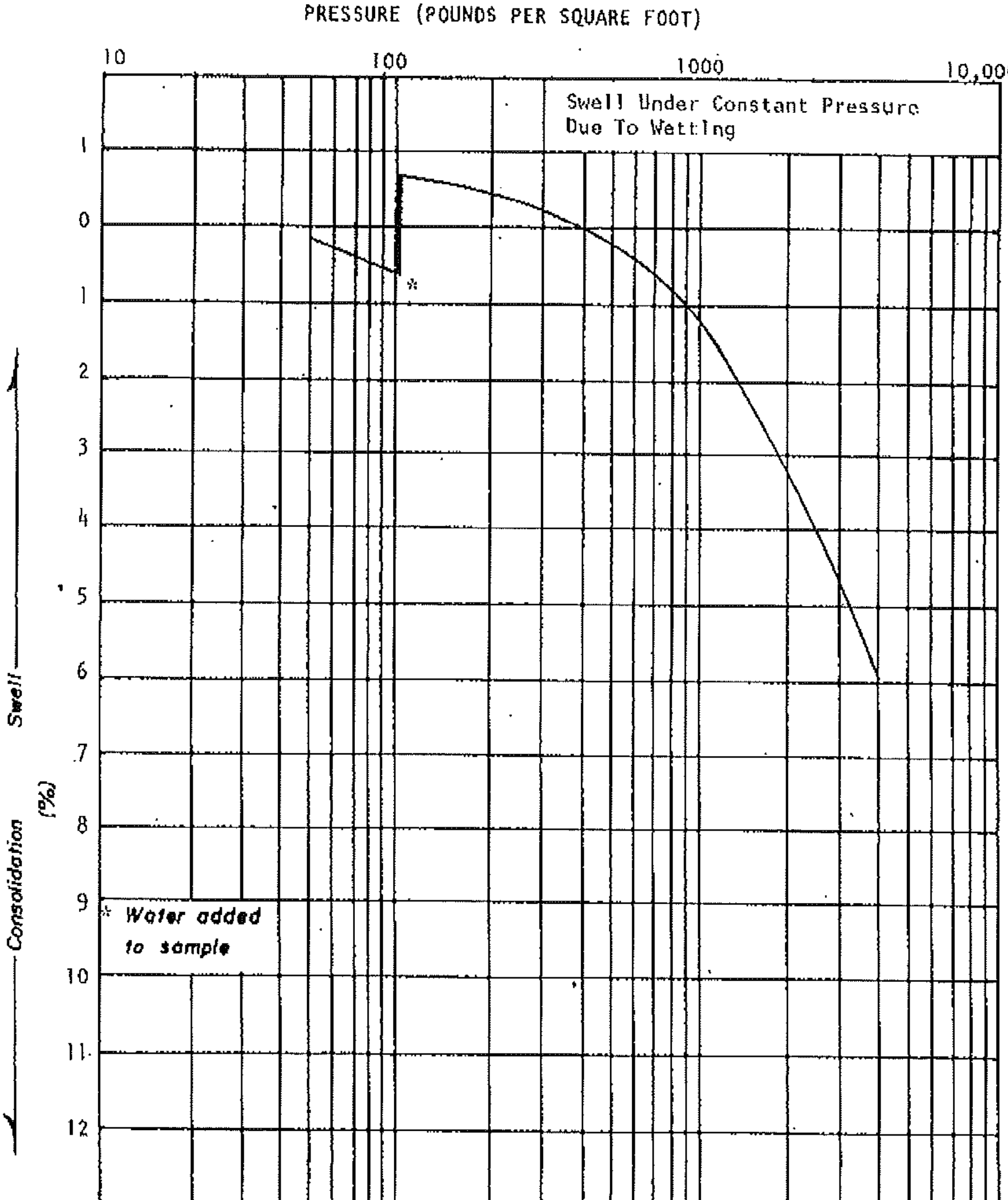
Chemical tests for water soluble sulfates and pH were performed on select samples obtained during the field study. The results of the chemical tests are tabulated below.

Test Boring	Depth (feet)	PH	Water Soluble Sulfates
1	1 to 4	7.2	greater than 160 ppm
5	1 to 4	7.1	greater than 160 ppm
6	1 to 4	7.3	greater than 160 ppm
9	1 to 4	6,9	greater than 160 ppm
10	1 to 4	7.2	greater than 160 ppm
12	1 to 4	10.7	greater than 160 ppm
13	1 to 4	7.0	greater than 160 ppm
19	1 to 4	6,9	greater than 160 ppm
22	1 to 4	7.2	greater than 160 ppm



Boring No. 1 Depth 4-5 feet	SUMMARY OF TEST RESULTS				
	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	9.7	90.0	1.0	1.94	400 ±
Final	22.7	110.0	.810	1.94	
Soil Description	Clay, silty, sandy, gravelly, brown				

SWELL - CONSOLIDATION TEST	
	Project No.: M03032GE
Lambert and Associates	Date: 3/13/03
	Figure: B1

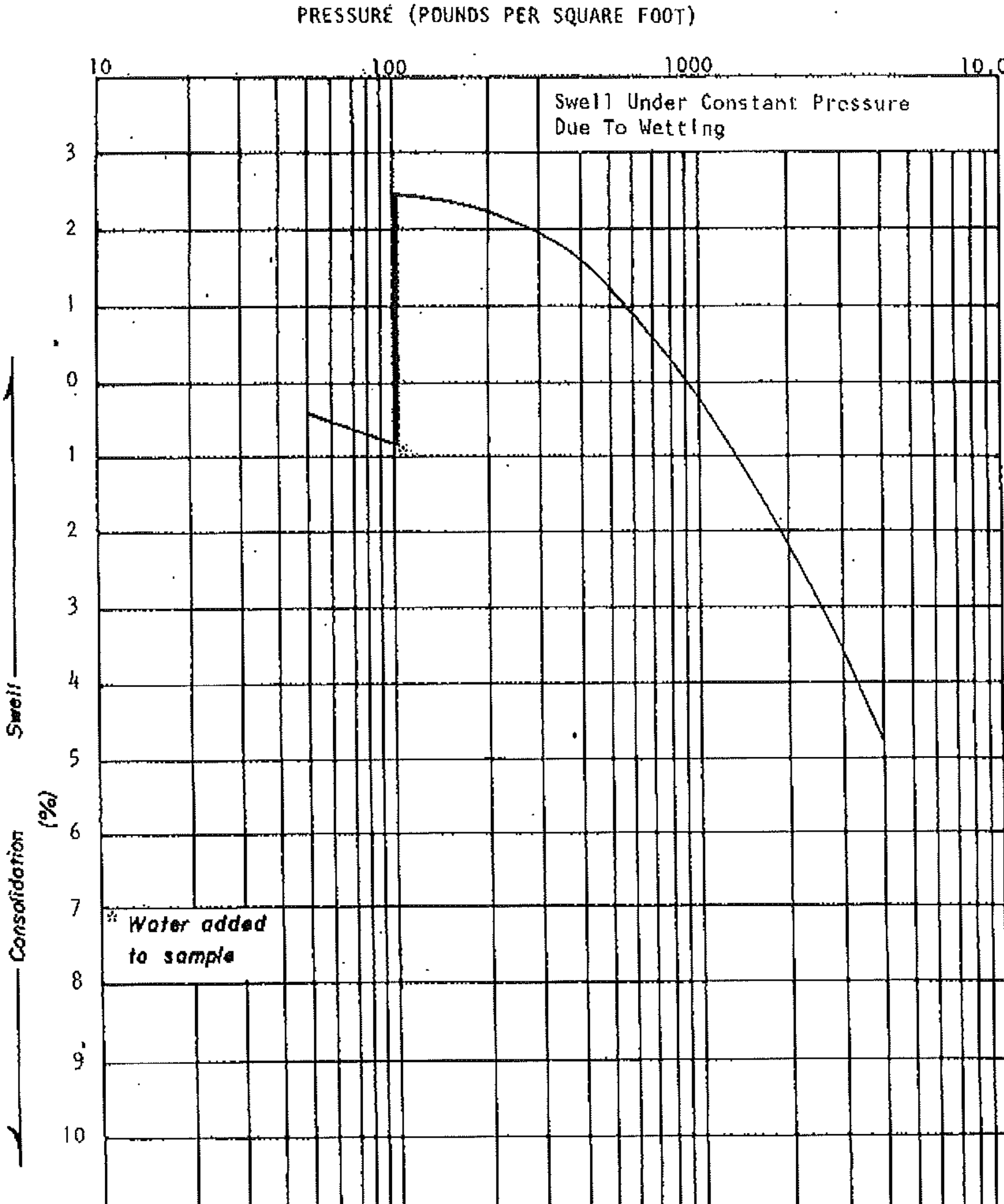


Boring No. 2	SUMMARY OF TEST RESULTS				
Depth 4-5 feet	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	10.6	108.0	1.0	1.94	
Final	21.3	113.0	.941	1.94	800 ±
Soil Description	Formational material, gray				

SWELL - CONSOLIDATION TEST

Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	B2

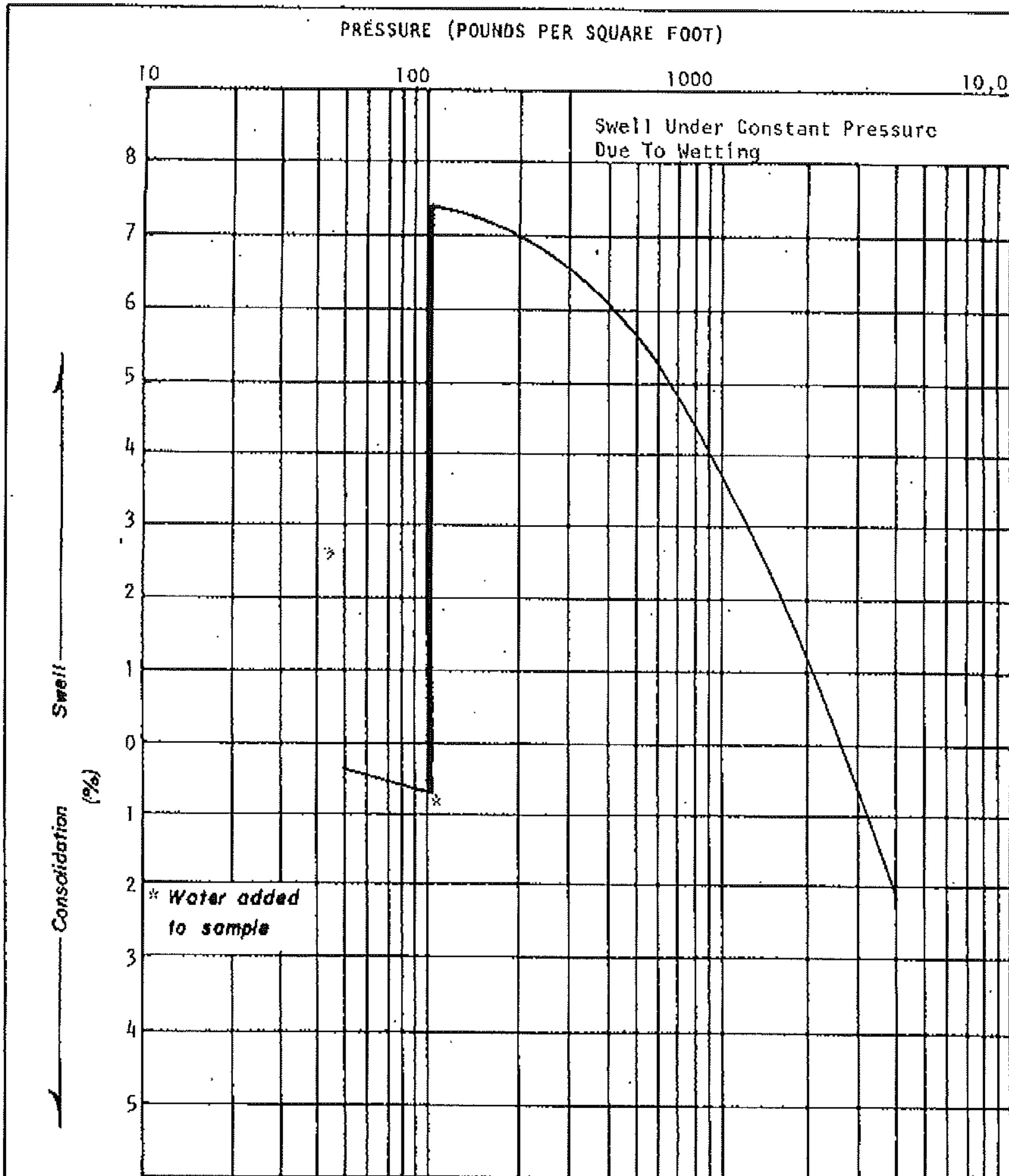


Boring No. 5 / Depth 4-5 feet	SUMMARY OF TEST RESULTS				
	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	12.3	117.0	1.0	1.94	1100 ±
Final	18.4	122.0	.953	1.94	
Soil Description	Formational material, gray				

SWELL - CONSOLIDATION TEST

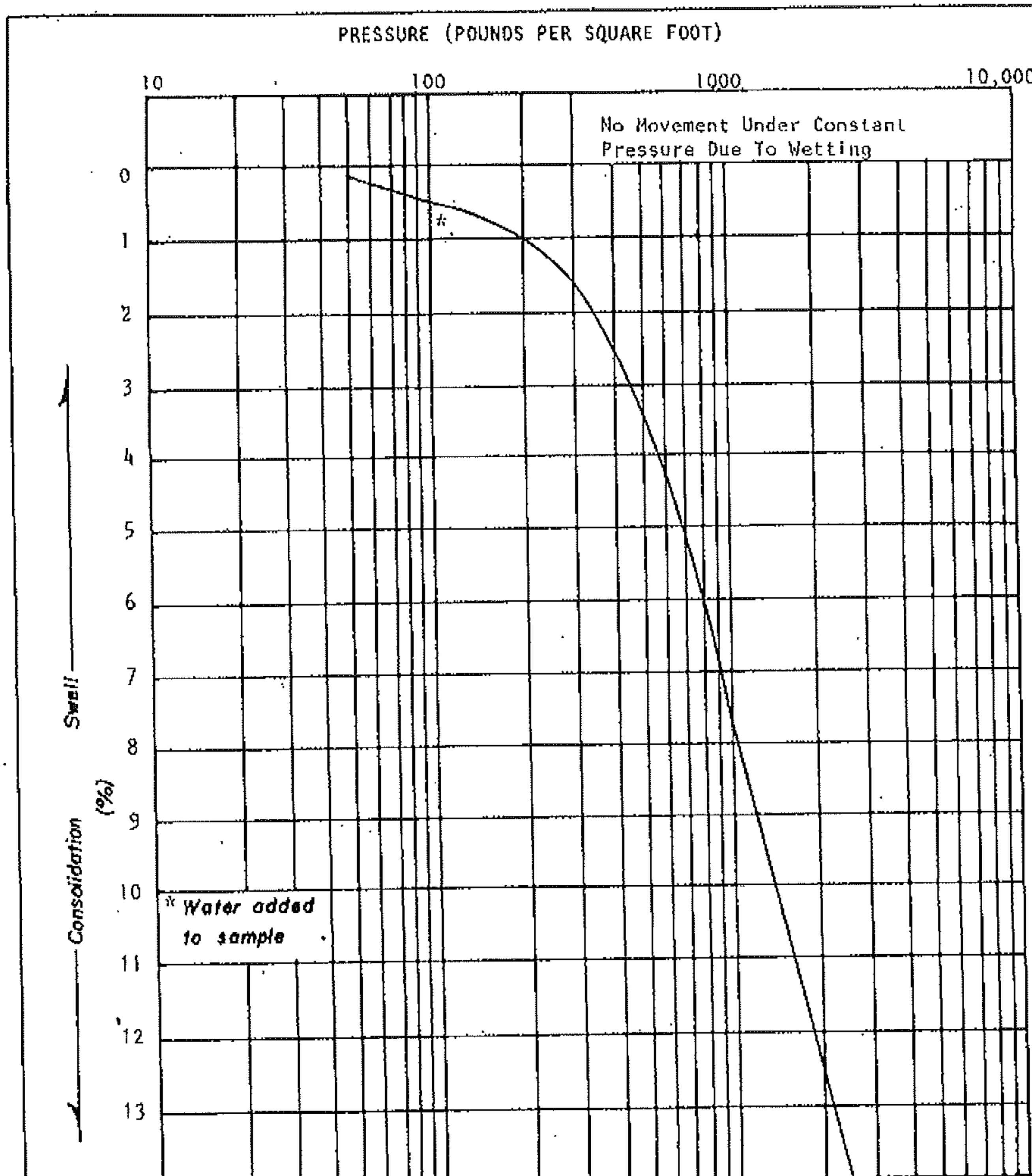
Lambert and Associates

Project No.: M03032GE
Date: 3/13/03
Figure: B3



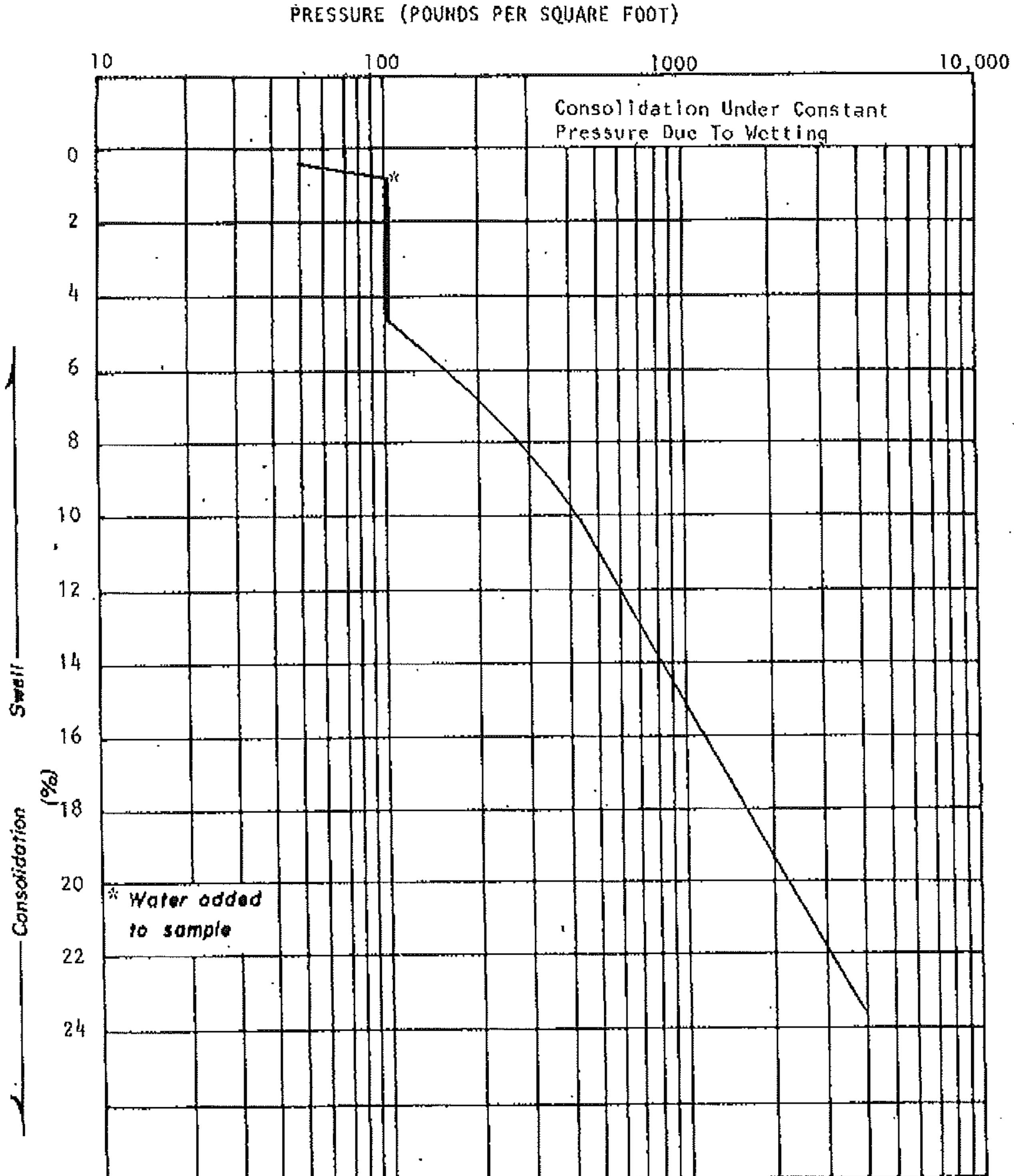
Boring No. 6 Depth 4-5 feet	SUMMARY OF TEST RESULTS				
	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	13.1	116.0	1.0	1.94	2600 ±
Final	20.2	118.0	.978	1.94	
Soil Description	Formational material, brown-gray				

SWELL - CONSOLIDATION TEST	Project No.: M03032GE
Lambert and Associates	Date: 3/13/03
	Figure: B4



Boring No. 9	SUMMARY OF TEST RESULTS				
Depth 4-5 feet	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	7.0	95.0	1.0	1.94	300 ±
Final	19.6	116.0	.821	1.94	
Soil Description	Clay, silty, slightly sandy, tan				

SWELL - CONSOLIDATION TEST	Project No.: H03032GE
Lambert and Associates	Date: 3/13/03
	Figure: B5

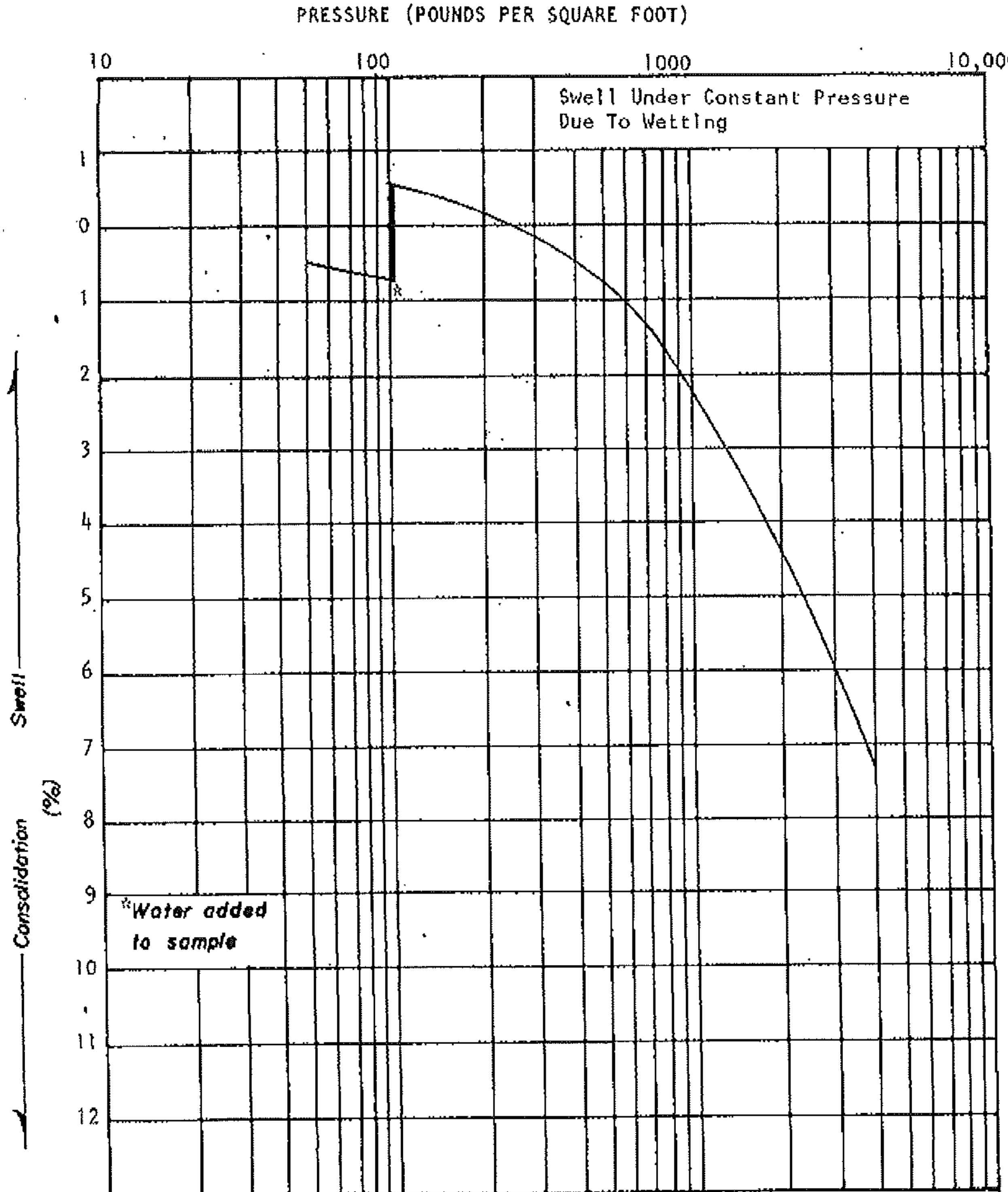


Boring No. 12 / Depth 5-6 feet	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
	SUMMARY OF TEST RESULTS				
Initial	7.2	87.0	1.0	1.94	less than 100
Final	19.6	114.0	.761	1.94	
Soil Description	Sand, gravel, cobbles, clayey, brown-gray-tan				

SWELL - CONSOLIDATION TEST

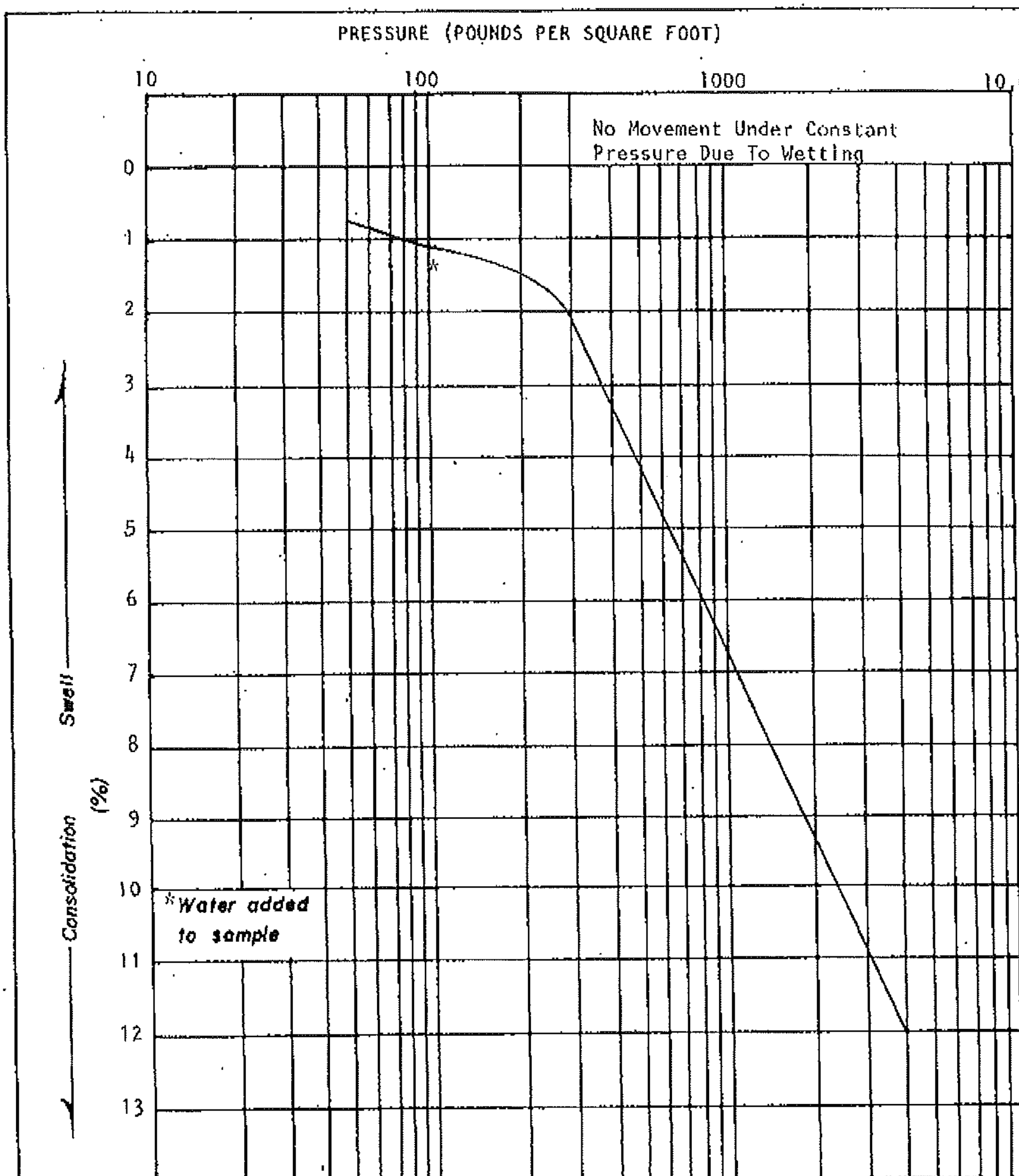
Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	B6



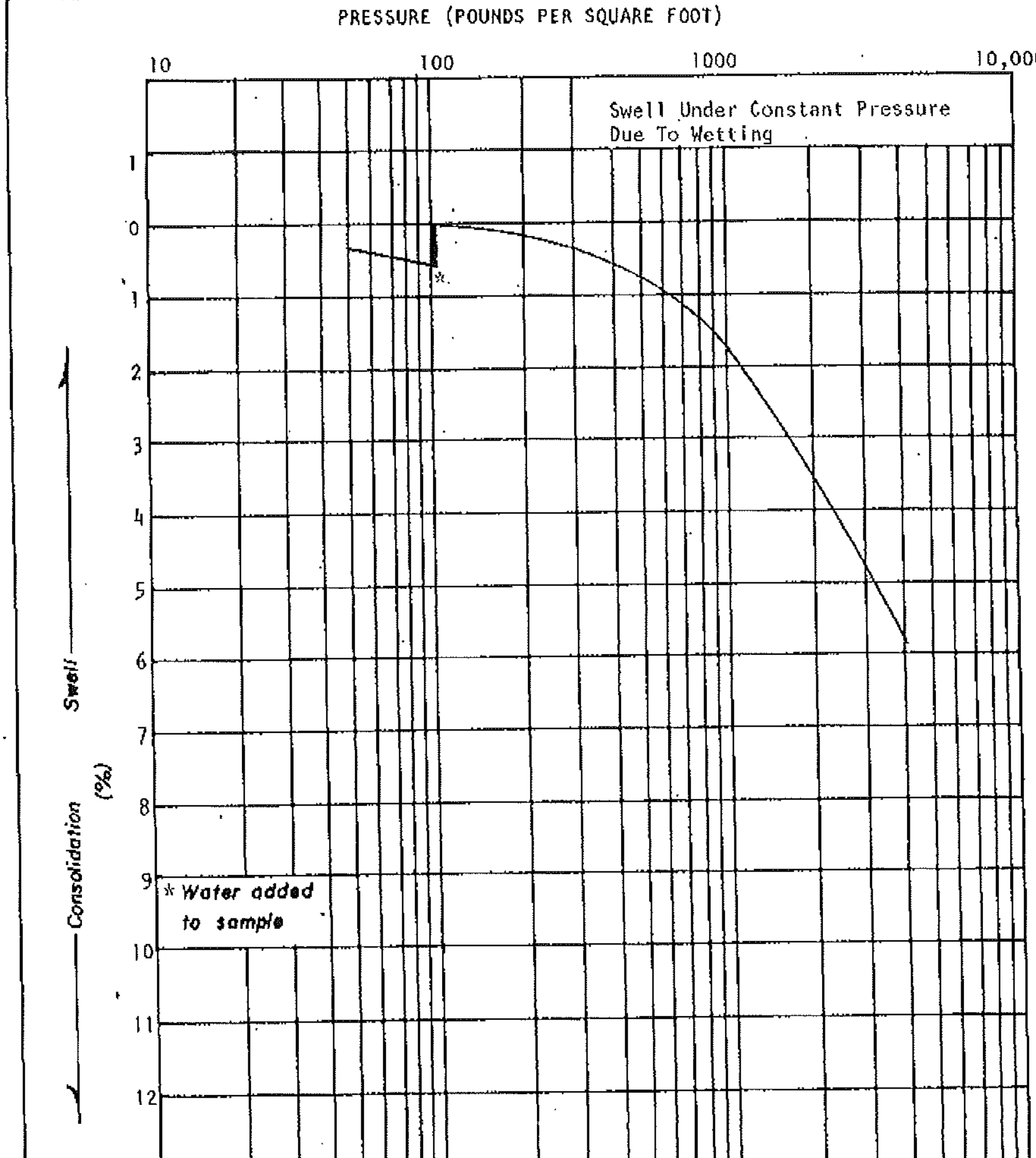
Boring No. 13 / Depth 4-5 feet	SUMMARY OF TEST RESULTS				
	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	10.9	106.0	1.0	1.94	600 ±
Final	22.4	114.0	.926	1.94	
Soil Description	Formational material, gray				

SWELL - CONSOLIDATION TEST	
	Project No.: M03032GE
Lambert and Associates	Date: 3/13/03
	Figure: B7



Boring No. 17	SUMMARY OF TEST RESULTS				
Depth 4-5 feet	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	9.7	103.0	1.0	1.94	200 ±
Final	17.8	116.0	.885	1.94	
Soil Description	Formational material, gray				

SWELL - CONSOLIDATION TEST	Project No.: M03032GE
Lambert and Associates	Date: 3/13/03
	Figure: B8

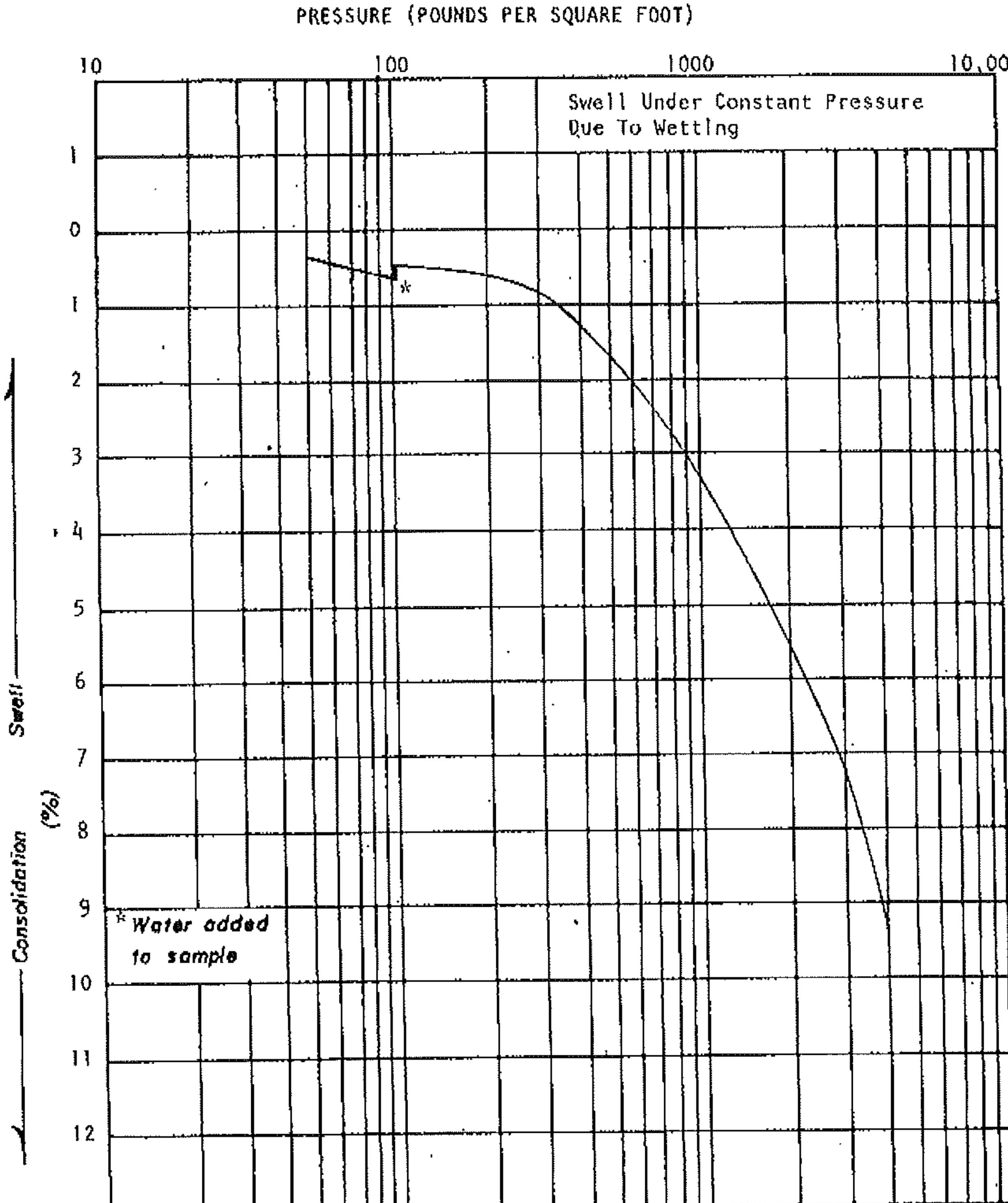


Boring No. 19 Depth 4-5 feet	SUMMARY OF TEST RESULTS				
	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	9.9	109.0	1.0	1.94	500 ±
Final	21.2	115.0	.940	1.94	
Soil Description	Formational material, gray				

SWELL - CONSOLIDATION TEST

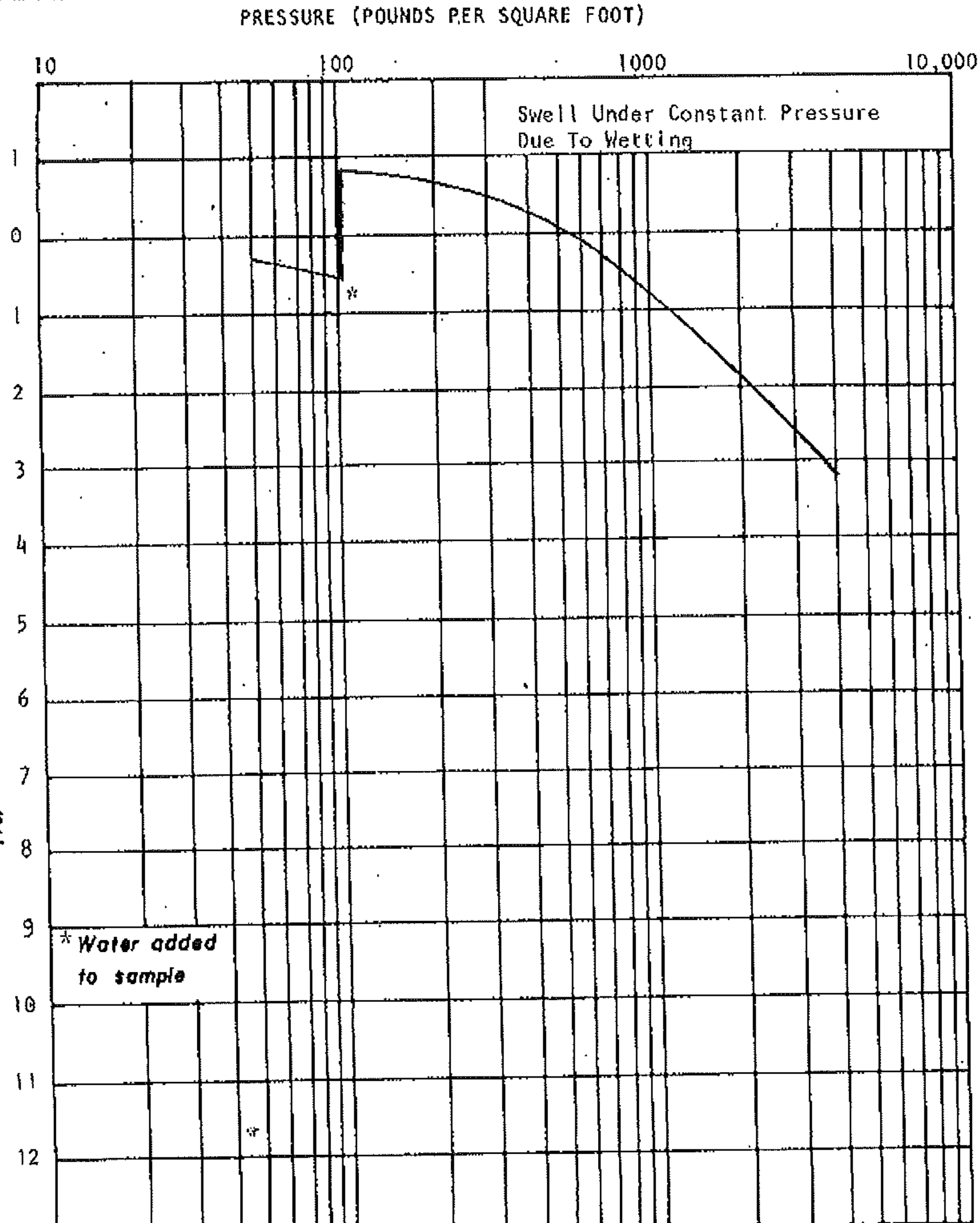
Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	B9



Boring No. 20 Depth 4-5 feet	SUMMARY OF TEST RESULTS				
	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	12.7	96.0	1.0	1.94	400 ±
Final	27.3	106.0	.901	1.94	
Soil Description	Clay, silty, brown				

SWELL - CONSOLIDATION TEST	Project No.: M03032GE
Lambert and Associates	Date: 3/13/03
	Figure: B10



SUMMARY OF TEST RESULTS

Boring No. 22 Depth 4-5 feet	Moisture Content (%)	Dry Density (P.C.F.)	Height (in.)	Diameter (in.)	Swell Pressure (P.S.F.)
Initial	8.4	124.0	1.0	1.94	800 ±
Final	15.5	140.0	.868	1.94	
Soil Description	Formational material, brown-brown gray				

SWELL - CONSOLIDATION TEST

Lambert and Associates

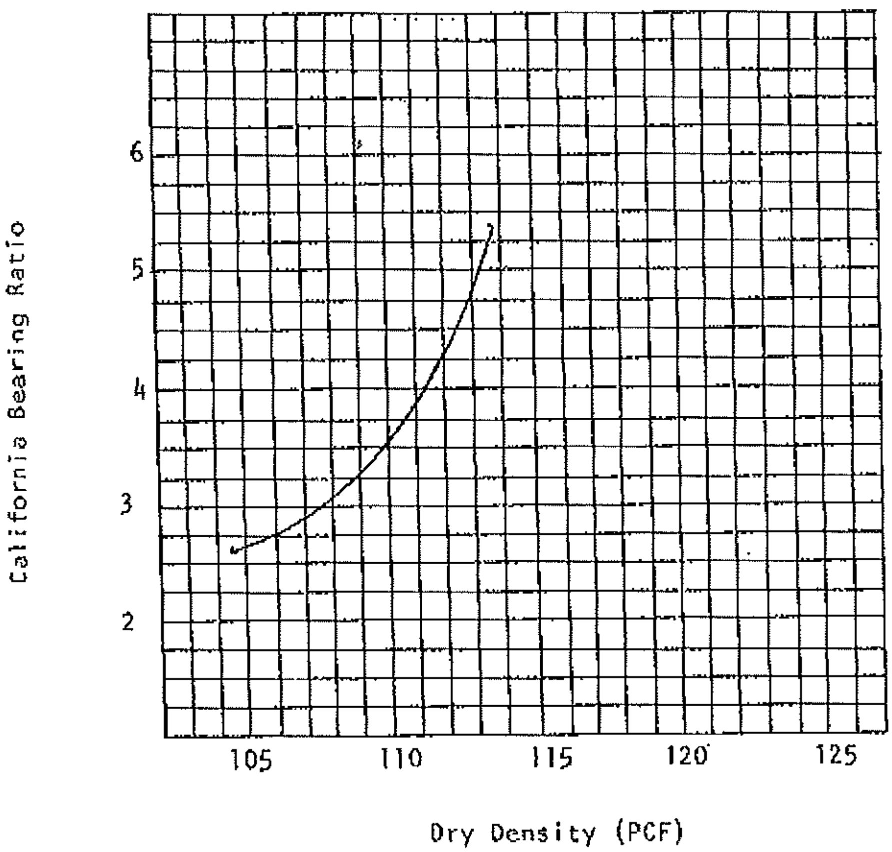
Project No.: M03032GE

Date: 3/13/03

Figure: B11

CALIFORNIA BEARING RATIO TEST RESULTS
ASTM D1883

Project Name: Parkerson Property Development Date: ____
Project Number: M03032GE Lab Number: 8848
Sample Decription: Clay, silty, brown Sample Location TB15 @ 1 to 4 feet & TB23 @ 1 to 5 feet blend
Proctor Method Used: ASTM D1557 Surcharge Weight: 15

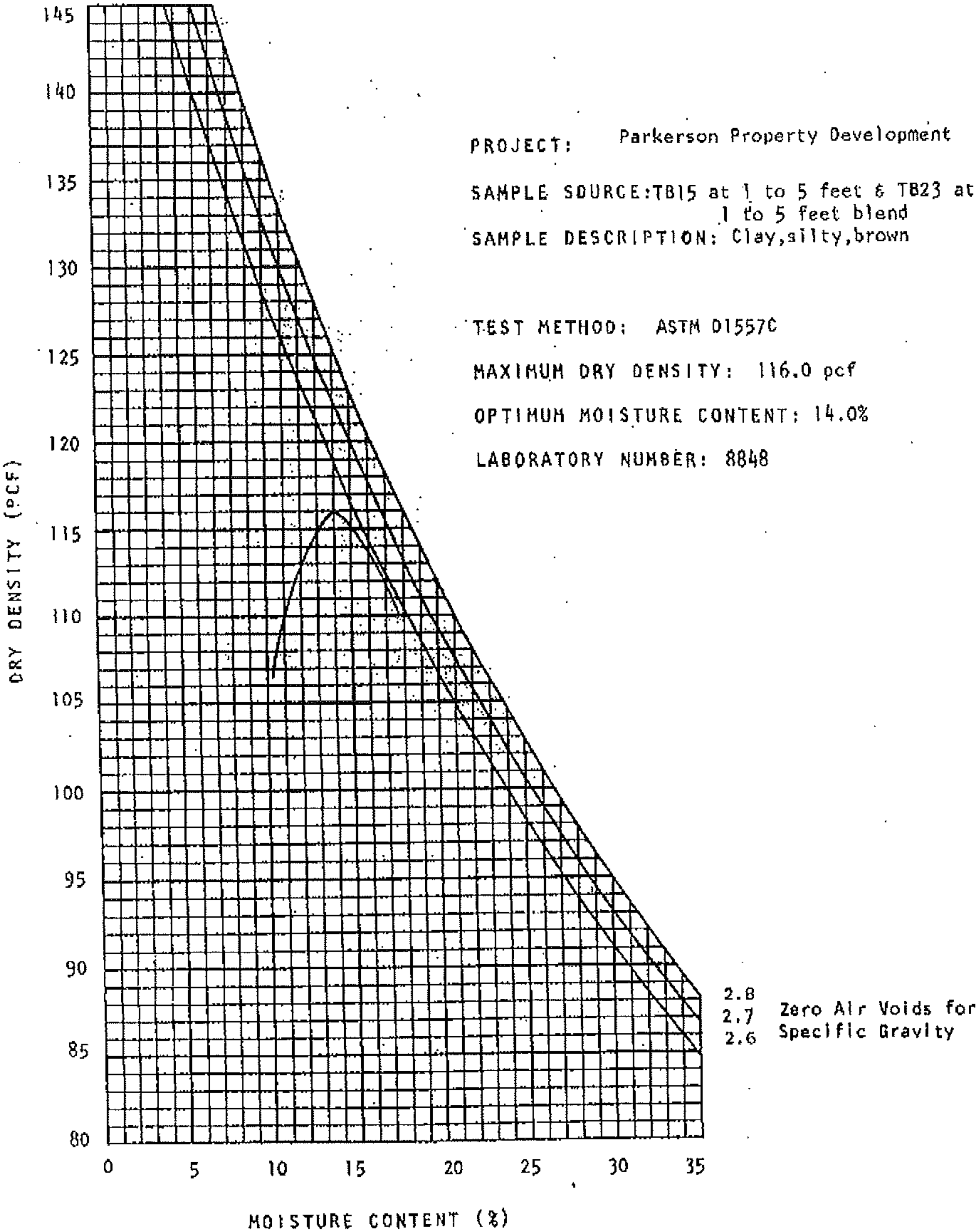


TEST DATA

PRE-SOAK		AFTER 96 HOUR SOAK			
Dry Density (pcf)	Moisture Content (%)	Dry Density (pcf)	Moisture Content Top One (1) Inch (%)	Swell (%)	CBR (%)
104.6	13.4	102.6	23.7	2.0	2.7
110.5	13.4	108.1	22.8	2.2	3.7
113.5	13.4	111.3	22.0	2.0	5.3

Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	B12



PROJECT: Parkerson Property Development

SAMPLE SOURCE: TB15 at 1 to 5 feet & TB23 at 1 to 5 feet blend

SAMPLE DESCRIPTION: Clay, silty, brown

TEST METHOD: ASTM D1557C

MAXIMUM DRY DENSITY: 116.0 pcf

OPTIMUM MOISTURE CONTENT: 14.0%

LABORATORY NUMBER: 8848

Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	B13

APPENDIX C

GENERAL GEOTECHNICAL ENGINEERING CONSIDERATIONS

C1.0 INTRODUCTION

Appendix C presents general geotechnical engineering considerations for design and construction of structures which will be in contact with soils. The discussion presented in this appendix are referred to in the text of the report and are intended as tutorial and supplemental information to the appropriate sections of the text of the report.

C2.0 FOUNDATION RECOMMENDATIONS

Two criteria for any foundation which must be satisfied for satisfactory foundation performance are:

- contact stresses must be low enough to preclude shear failure of the foundation soils which would result in lateral movement of the soils from beneath the foundation, and

- settlement or heave of the foundation must be within amounts tolerable to the superstructure.

The soils encountered during our field study have varying engineering characteristics that may influence the design and construction considerations of the foundations. The characteristics include swell potential, settlement potential, bearing capacity and the bearing conditions of the soils supporting the foundations. The general discussion below is intended to increase the readers familiarity with characteristics that can influence any structure.

C2.1 Swell Potential

Some of the materials encountered during our field study at the anticipated foundation depth may have swell potential. Swell potential is the tendency of the soil to increase in volume when it becomes wetted. The volume change occurs as moisture is absorbed into the soil and water molecules become attached to or adsorbed by the individual clay platlets. Associates with the process of volume change is swell pressure. The swell pressure is the force the soil apples on its surroundings when moisture is absorbed into the soil. Foundation design considerations concerning swelling soils include structure tolerance to movement and dead load pressure to help restrict uplift. The structure's tolerance to movement should be addressed by the structural engineer and is dependent upon many facets of the design including the overall structural concept and the building material. The uplift forces or pressure due to wetted clay soils can be addressed by designing the foundations with a minimum dead load and/or placing the foundations on a blanket of compacted structural fill. The compacted structural fill blanket will increase the dead load on the swelling foundation soils and will increase the separation of the foundation from the swelling soils. Suggestions and recommendations for design dead load and compacted structural fill blanket are presented below. Compacted structural fill recommendations are presented under COMPACTED STRUCTURAL FILL below.

C2.2 Settlement Potential

Settlement potential of a soil is the tendency for the soil to experience volume change when subjected to a load. Settlement is characterized by downward movement of all or a portion of the supported structure as the soil particles move closer together resulting in decreased soil volume. Settlement potential is a function of;

. foundation loads,
. depth of footing embedment,
. the width of the footing, and
. the settlement potential or compressibility of the influenced soil.

Foundation design considerations concerning settlement potential include the amount of movement tolerable to the structure and the design and construction concepts to help reduce the potential movement. The settlement potential of the foundation can be reduced by reducing foundation pressures and/or by placing the foundations on a blanket of compacted structural fill. The anticipated post construction settlement potential and suggested compacted fill thickness recommendations are based on site specific soil conditions and are presented in the text of the report.

C2.3 Soil support Characteristics

The soil bearing capacity is a function of;

. the engineering properties of the soil material supporting the foundations,
. the foundation width,
. the depth of embedment of the bottom of the foundation below the lowest adjacent grade,
. the influence of the ground water, and
. the amount of settlement tolerable to the structure.

Soil bearing capacity and associated minimum depth of embedment are presented in the text of the report.

The foundation for the structure should be placed on relatively uniform bearing conditions. Varying support characteristics of the soils supporting the foundation may result in nonuniform or differential performance of the foundation. Soils encountered at foundation depths may contain cobbles and boulders. The cobbles and boulders encountered at foundation depths may apply point loads on the foundation resulting in nonuniform bearing conditions. The surface of the formational material may undulate throughout the building site. If this is the case, it may result in a portion of the foundation for the structure being placed on the formational material and a portion of the foundation being placed on the overlying soils. Varying support material will result in nonuniform bearing conditions. The influence of nonuniform bearing conditions may be reduced by placing the foundation members on a blanket of compacted structural fill. Suggestions and recommendations for constructing compacted structural fill are presented under COMPACTED STRUCTURAL FILL below and in the text of the report

C3.0 COMPACTED STRUCTURAL FILL

Compacted structural fill is typically a material which is constructed for direct support of structures or structural components.

There are several material characteristics which should be examined before choosing a material for potential use as compacted structural fill. These characteristics include;

. the size of the larger particles,
. the engineering characteristics of the fine grained portion of material matrix,
. the moisture content that the material will need to be for compaction with respect to the existing initial moisture content, the organic content of the material, and
. the items that influence the cost to use the material.

Compacted fill should be a non-expansive material with the maximum aggregate size less than about two (2) inches and less than about twenty five (25) percent coarser than three quarter (3/4) inch size.

The reason for the maximum size is that larger sizes may have too great an influence on the compaction characteristics of the material and may also impose point loads on the footings or floor slabs that are in contact with the material. Frequently pit-run material or crushed aggregate material is used for structural fill material. Pit-run material may be satisfactory, however crushed aggregate material with angular grains is preferable. Angular particles tend to interlock with each other better than rounded particles.

The fine grained portion of the fill material will have a significant influence on the performance of the fill. Material which has a fine grained matrix composed of silt and/or clay which exhibits expansive characteristics should be avoided for use as structural fill. The moisture content of the material should be monitored during construction and maintained near optimum moisture content for compaction of the material.

Soil with an appreciable organic content may not perform adequately for use as structural fill material due to the compressibility of the material and ultimately due to the decay of the organic portion of the material.

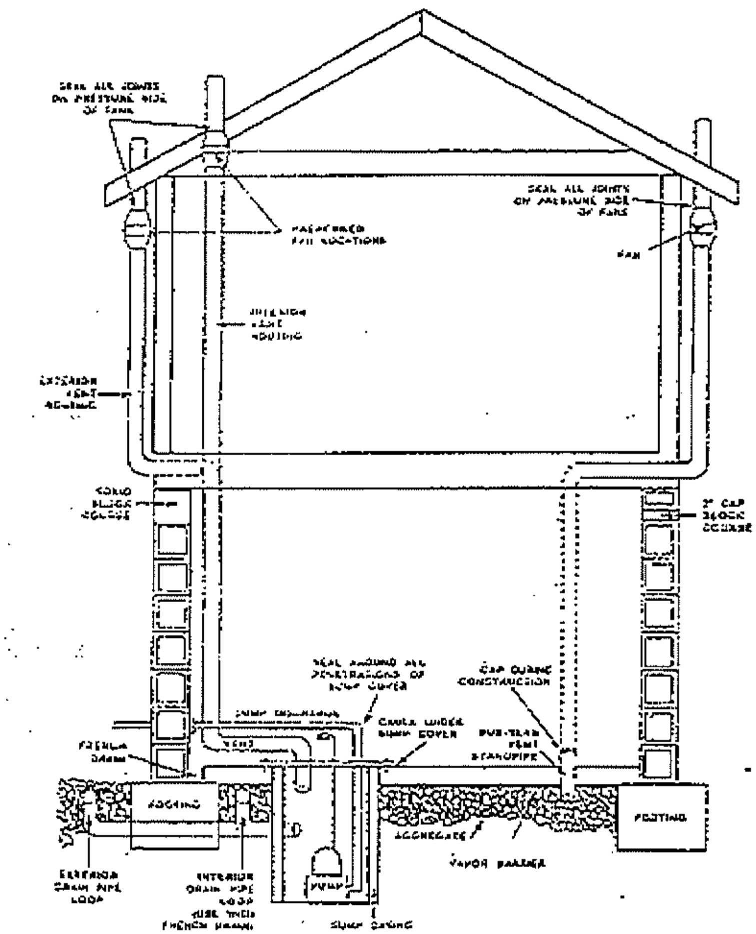
C4.0 RADON CONSIDERATIONS

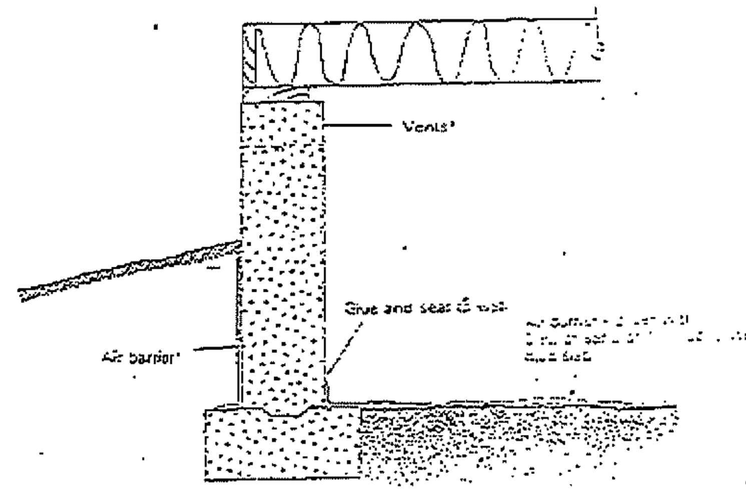
Information presented in "Radon Reduction in New Construction, An Interim Guide: OPA-87-009 by the Environmental Protection Agency dated August 1987" indicates that currently there are no standard soil tests or specific standards for correlating the results of soil tests at a building site with subsequent indoor radon levels. Actual indoor levels can be affected by construction techniques and may vary greatly from soil radon test results. Therefore it is recommended that radon texts be conducted in the structure after construction is complete to verify the actual radon levels in the home.

We suggest that you consider incorporating construction techniques into the development to reduce radon levels in the residential structures and provide for retrofitting equipment for radon gas removal if it becomes necessary.

Measures to reduce radon levels in structures include vented crawl spaces with vapor barrier at the surface of the crawl space to restrict radon gas flow into the structure or a vented gravel layer with a vapor barrier beneath a concrete slab-on-grade floor to allow venting of radon gas collected beneath the floor and to restrict radon gas flow through the slab-on-grade floor into the structure. These concepts are shown on Figure C1.

If you have any questions or would like more information about radon. Please contact us or the State Health Department at 303-692-3030.





Notes:

¹ Cross-laminated or 6-mil polyethylene applied at basement wall construction.

² Provide 25% of vent area to face in each direction. Vents may be placed on three sides positioned as close as possible to building corners. Maximum aggregate vent size should not exceed the 0.5% of total square feet of crawl space floor area

METHODS TO FACILITATE POST-CONSTRUCTION RADON REMOVAL

These figures were excerpted from an EPA manual "Radon Reduction in New Construction, An Interim Guide" OPA-87-009 by the Environmental Protection Agency, and reproduced here for reference only

RADON FLOW CONCEPT

Lambert and Associates

Project No.:	M03032GE
Date:	3/13/03
Figure:	C1