REPORT FOR
GEOTECHNICAL ENGINEERING SERVICES
DYSART DRAIN IMPROVEMENTS PROJECT
CONTRACT FCD 93-01
REEMS ROAD TO THE AGUA FRIA RIVER
MARICOPA COUNTY, ARIZONA

Registered Professional Engineer (Civil)
CERTIFICATE NO.
24033
KENNETH D.
WALSH
Date Signed 2/1/94
ARIZONA, U.S.A.

THOMAS-HARTIG & ASSOCIATES, INC.

GEOTECHNICAL, MATERIALS TESTING, AND ENVIRONMENTAL CONSULTANTS

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REPORT FOR GEOTECHNICAL ENGINEERING SERVICES DYSART DRAIN IMPROVEMENTS PROJECT CONTRACT FCD 93-01 REEMS ROAD TO THE AGUA FRIA RIVER MARICOPA COUNTY, ARIZONA

Submitted To:

NBS/Lowry Engineers & Planners
Attention: Brian Fry, P.E.
2600 North 44th Street, Suite 200
Phoenix, Arizona 85008-1599

Project No. 93-1033

28 January 1994

THOMAS-HARTIG & ASSOCIATES, INC.

Geotechnical, Materials Testing, and Environmental Consultants

7031 West Oakland Street • Chandler, Arizona 85226

Chandler: Phone (602) 961-1169, Fax (602) 940-0952 • Phoenix: Phone (602) 437-5450

Tom W. Thomas, P.E.
James R. Morrow
John P. Boyd, P.E.
Kenneth L. Ricker, P.E.

Charles H. Atkinson, P.E.
James M. Willson, P.E.
Steven A. Haire, P.E.

Judith A. McBee
Dale V. Bedenkop, P.E.
John C. Patton
Kenneth D. Walsh, P.E.

NBS/Lowry Engineers & Planners
2600 North 44th Street, Suite 200
Phoenix, Arizona 85008-1599

28 January 1994

Attention: Brian Fry, P.E.

Subject: Report of Geotechnical Engineering Services — Project No. 93-1033
Dysart Drain Improvements Project
Contract FCD 93-01
Reems Road to the Agua Fria River
Maricopa County, Arizona

This report presents the results of the geotechnical engineering services authorized on the site for the proposed Dysart Drain Improvements Project located in Maricopa County, Arizona.

The purpose of these services is to determine the soil conditions at the locations indicated which thereby provide a basis for the design discussions and recommendations presented herein. This firm should be notified for evaluation if conditions other than described herein are encountered during construction.

The services performed provide an evaluation at selected locations of the soils throughout the zone of significant foundation influence. Our field services have not included exploration for underlying geologic conditions or evaluation of potential geologic hazards such as seismic activity, faulting, and ground subsidence/cracking potential due to groundwater withdrawal, or the presence of contamination.

The recommendations presented in this report are based upon the project information received and described in "Scope" Part I. This firm should be contacted for review if the design conditions are changed substantially.

If requested, we will be available to review project plans and specifications relative to compliance to the intent of this report.

Respectfully submitted,

THOMAS-HARTIG & ASSOCIATES, INC.

Registered Professional Engineer (Civil) CERTIFICATE NO. 24033 KENNETH D. WALSH Date Signed 2/1/94 ARIZONA, U.S.A.

By:

Reviewed By: .

Registered Professional Engineer (Civil) CERTIFICATE NO. 10971 KENNETH L. RICKER Date Signed 2/1/94 ARIZONA, U.S.A.

/cm

Copies to: Addressee (5)

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PART I

REPORT

SCOPE

The proposed Dysart Drain Improvement Project includes the construction of a new lined channel along the alignment of the existing lined and earth channel from Reems Road to the Agua Fria River. The channel invert will be lowered to correct past and projected future subsidence and to accommodate a larger design flow, and will be up to nearly 30 feet deep. A detention basin will be constructed northwest of Luke Air Force Base (LAFB) at the northeast corner of Reems Road and Northern Avenue to intercept flows north and west of the Base and direct them into the drain. Incidental improvements include three new bridge crossings and associated pavements and new box culverts. This report contains a description of our field operations and laboratory results and design recommendations concerning constructability, excavations and slope stability, bridge foundations, bearing capacity and lateral earth pressures, bedding and backfilling materials, and pavement thicknesses for cross streets affected by this project.

SITE DESCRIPTION

The Dysart Drain area from Reems Road to the Agua Fria River consists of an incised open channel with undercrossings at Dysart Road, the Morton Salt facility, and El Mirage Road. The canal is lined east of the AT & SF Railroad crossing, but unlined west of the AT & SF Railroad crossing. It follows an alignment roughly adjacent to Northern Avenue to a point approximately in line with the LAFB runway, and turns southeast at this point near the intersection with the AT& SF railroad. Just east of Litchfield Road, the channel turns east and continues to the Agua Fria River roughly along the midpoint line between Northern and Glendale Avenues. Unpaved service roads are provided along one or both banks. The area traversed by the channel is largely agricultural, except for LAFB and associated residential areas and the Morton Salt and Amerigas facilities east of Dysart Road. East of Morton Salt the surrounding area is vacant land. During our field operations, interim repairs to the negative channel slope created by past differential subsidence along the alignment were being conducted east of Dysart Road. The channel carried small flows from rain storms in the area intermittently throughout our field operations.

INVESTIGATION

The field investigation included a site reconnaissance, subsurface exploration, and field percolation testing. The subsurface exploration consisted of drilling 46 test

borings at the locations shown on the site plan in Appendix A. The test borings were drilled with a CME-75 drill rig using 7-inch diameter hollow stem augers. The test borings were drilled to depths ranging from 10 to 80 feet. Standard Penetration Test (SPT) sampling and 2.42-inch ring-lined driven soil sampling was performed in all borings, alternating at 5- foot intervals, to obtain an indication of the relative density and/or consistency of the formation being penetrated and to obtain samples for laboratory testing. Previous test borings (Appendix A) performed by Speedie and Associates for the interim repairs are also shown on the Site Plan in Appendix A. New test borings adjacent to Speedie and Associates test borings were only sampled below the deepest penetration of the previous test borings. In addition, bulk samples were obtained from the auger cuttings. Seven shallow subgrade samples were obtained in proposed pavement reconstruction areas (A through G). Percolation tests were conducted in three test borings (P-1, P-2 and P-3) in the proposed detention basin area at depths of 12, 14, and 16 feet, respectively.

During the field investigation, the soils encountered were visually classified by our field engineer. The results of the test drilling conducted for this project are presented on the boring logs in Appendix A, "Field Results."

LABORATORY INVESTIGATION

Laboratory testing was conducted on representative soil samples obtained during the test drilling. The testing was conducted to obtain the data necessary to develop design recommendations for this project. The following tests were conducted:

Test	Sample(s)	Purpose
Sieve Analysis & Atterberg Limits	Representative (30)	Classification and correlation of engineering properties
Dry density and Moisture Content	Undisturbed (104) Disturbed (114)*	In-situ density and moisture determination to correlate engineering properties

Test	Sample(s)	Purpose
Direct Shear	Undisturbed (13)	Bearing capacity, drilled shaft capacity, and slope stability analysis
Consolidation	Undisturbed (5)	Settlement analyses
Soluble Salts, Sulfates and Chlorides	Representative (4)	Corrosion potential
pH and Resistivity	Representative (4)	Corrosion potential
ASTM D698	Representative Grab Sample (6)	Compaction characteristics
R-value	Representative Grab Sample (4)	Pavement design
Expansion	Compacted (10) Undisturbed (1)	Expansion potential

*Disturbed samples from SPT sampling tested for moisture content only.

The results of the moisture and density testing are presented on the graphical boring logs in Appendix A. The results of the remainder of the testing are presented in Appendix B.

SOIL CONDITIONS

The soils encountered along the proposed channel alignment generally consisted of interbedded, stratified sandy clays and silty sands. The clay materials are generally stiff to hard, although thin, very hard cemented zones were encountered in some areas typically below a depth of 10 to 15 feet. The plasticity of the clays ranged from low to high. Low plasticity clays were common at the surface and the western portion of the alignment. High plasticity clays were encountered along much of the center portion of the alignment, typically in a 5 to 10 feet thick zone about 20 feet below ground. Seams or layers of clean to silty subangular to subrounded sands with gravels were observed in scattered locations in the test borings, ranging from a few inches to a few feet in thickness. The sandy soils became much coarser and continuous east of El Mirage Road and were generally dense to very dense with some zones containing cobbles in the vicinity of the Agua Fria River. Some test borings in the channel bottom inside Luke Air Force Base

encountered clean medium to coarse sands in the upper six to twelve inches. Soil moisture conditions generally ranged from damp to moist, and no free groundwater was encountered in the test borings during drilling. Groundwater conditions may vary with time, seasonal conditions, and/or flow in the drain or the Agua Fria River. Graphical boring logs for all test borings are presented in Appendix A.

The drainage channel crosses an active earth fissure area on the east side of the Morton Salt Facility. The existing lined channel in this area shows some cracking but no holes, voids or large cracks exist in the channel lining nor have any been reported in the past.

DISCUSSION AND RECOMMENDATIONS

General: Geotechnical engineering recommendations are presented in the following sections. These recommendations are based upon the results of the field and laboratory testing and calculations which are presented in Appendices A, B, and C of this report. Alternative recommendations may be possible and will be considered upon request.

Because the site contains an existing drainage structure and other improvements including utilities, special attention should be paid to locating and removing any subsurface remains of past development that may exist in the proposed channel and drainage structure areas, such as buried utilities, concrete slab remnants, drainage structure remnants, loose or water softened soils, disturbed soil zones, etc. Because of the potential for such problems, the cleared site and all channel and structure excavations should be observed by the engineer or a representative of the engineer to verify removal of debris and competence of bearing soils. Even if the site reconstruction is closely monitored, the possibility, although remote, does exist that some buried fills, undiscovered facility remnants, etc., may remain undetected. Any such conditions would be expected to be localized and cause only local distress to the proposed improvements.

Expansion Potential: The native sandy clay surface soils were generally of low to medium plasticity, but high plasticity soils are present, typically in the vicinity of the proposed channel invert. At existing moisture and density conditions, the low plasticity soils would probably demonstrate slight expansive potentials. These potentials will be increased if soils are compacted to relatively high density at

moisture contents below optimum. However, the high plasticity soils were found to be very expansive when wetted, even at existing moisture and density conditions. The expansive potentials of fills constructed using these high plasticity clay soils are estimated on the order of over one inch per foot of compacted soil. For this reason, highly plastic soils removed from the excavation should not be used in any required structural fills or backfills. Further, the high plasticity soils should be removed from within two feet of the bottom or sides of the channel lining, and replaced with low plasticity clayey sand or silty sand soils as described in Part II of this report. Further, the transition from the channel bottom to the channel sides should be reinforced to better distribute stresses in the event of any movements.

Channel Configuration

From the results of shear strength testing on undisturbed samples, we have performed an analysis of slope configurations along the proposed channel. Based on this evaluation, in our opinion lined channel slopes no steeper than 1.5H:1V will be stable within the clay soils encountered west of El Mirage Road. Occasional granular seams and layers encountered in the test borings may experience ravelling or sloughing on the face of the channel until the lining is completed at this slope, a condition which could be largely eliminated by flattering to 2H:1V or flatter. If the upper portions of the channel are to be unlined, we recommend that at least a 4 foot wide bench be placed at the top of the lined section. The maintenance roads and/or unused right-of-way should be graded or provided with interceptor channels to prevent surface runoff over and down the face of the unlined upper portion. Even with these precautions, erosion of the unlined channel facing should be expected. Maintenance of the unlined channel face and periodic removal of eroded sediment from the bench or the channel will be necessary, and will be more frequent if surface water runoff is allowed to flow over and down the exposed face. In the quarter mile west of El Mirage Road, sandy soils are typically located within about 10 feet of the surrounding ground surface. In this reach, the unlined portion should either end at the bottom of the clay surface layer (about 10 feet deep), or the upper unlined portion should have a slope no steeper than 2H:1V.

East of El Mirage Road, soils are generally granular beginning at or very near the ground surface. Therefore, for the reach between El Mirage and the Agua Fria River, both the upper unlined portion and the lower lined portion should be no

steeper than 2H:1V. The minimum 4 foot wide bench at the top of the lined portion and the top of bank drainage should be provided in this reach as well.

The removal of the existing channel lining and excavation of the new channel in the active earth fissure area on the east side of the Morton Salt facility should be closely observed for existing fissures. If encountered, the open fissures should be overexcavated to a depth of 3 feet and for a width of at least 3 feet on either side of the fissure. The open fissure below this zone should be backfilled with aggregate base. A 10 mil PVC membrane and a geogrid should be placed in the bottom of the overexcavated area and the overexcavated zone backfilled with soil having a plasticity index of at least 12 and a minus No. 200 Sieve of at least 60 percent. This soil should be placed at to slightly above optimum moisture content and compacted to 100 percent of the ASTM D698 maximum dry density.

As reported earlier, the existing channel liner has not undergone severe distress as the result of earth fissuring in the area; therefore, it is our opinion that additional precautions or special drainage systems such as a double liner or additional reinforcements are not warranted for this project.

Foundations: Wing walls, box culverts, and/or retaining walls founded at shallow levels below the finished grade and supported on compacted fill and/or undisturbed native soils were evaluated for support of the proposed improvements. The soils at the box culvert sites are moderately compressible, and the box culverts will be lighter than the soil they replace. Therefore, we anticipate low settlements of less than 1/2 inch. The following tabulation presents foundation bearing design recommendations for shallow wall or box culvert foundations bearing on undisturbed native soils and/or compacted fills.

Foundation Depth	Allowable Foundation Bearing Pressure
1.5 ft.	2000 psf
2 ft.	2250 psf
3 ft.	2500 psf
4 ft.	2750 psf

Foundation depth refers to the depth of the foundation base below finished grade, which is defined as the lowest adjacent grade within 5 feet of the foundation for wingwalls and retaining walls or final channel grade for box culverts.

Foundation bearing surfaces should be free of debris and water softened materials prior to placing concrete and reinforcing steel. All foundation excavations should be reviewed by the geotechnical engineer prior to placing the foundation material. Any loose or disturbed zones should be removed and replaced with compacted fill or lean concrete.

Recommended foundation bearing pressures should be considered allowable maximums for dead load plus design live loads and may be increased by one-third when considering transient wind or seismic forces. The weight of foundation concrete below grade may be neglected in dead load computations. All wall foundations should have a minimum width of 1.33 feet.

It is anticipated that the bridge replacements at Dysart Road and El Mirage Road and Morton Salt will be single span structures supported on stub abutments behind the channel lining. Based on available data, we recommend that the bridge abutments be supported on 36 to 60-inch diameter machine cleaned, straight shaft drilled piles founded below the adjacent proposed channel bottom. The capacities of the various lengths and diameters of drilled piles for the Dysart Road and El Mirage Road crossings are presented on the charts in Appendix D. Foundation recommendations for the Morton Salt Bridge will be included upon completion of laboratory testing. The finish grade indicated on the vertical axis of the chart should be taken as the elevation of the bottom of the channel. The presented allowable drilled pile capacities should be considered allowable maximums for dead plus design live loads, and may be increased by one-third when considering total loads including transient wind or seismic forces. The weight of the foundation concrete below grade may be neglected in dead load computations. Settlements for the anticipated foundation loads are less than about 1/2 inch. Disturbed soil or drilling spoil must be removed from the bottom of all drilled pile excavations, and might be removed satisfactorily by machine (drill rig) cleaning. If adequate cleaning cannot be accomplished by machine cleaning, then manual cleaning will be required. Applicable safety codes will require safety casing for protection if personnel enter drilled pier excavations for cleaning or observation purposes. A

minimum pile diameter of 3 feet is recommended, as smaller augers sometimes have difficulty removing the larger cobbles in the sand and gravel deposits with cobbles encountered, particularly in the vicinity of the Agua Fria River. Piles should be spaced no closer than 2.5 diameters, center to center.

Moderate caving is probable for drilled pile excavations in the sandy soils. Severe caving is most likely if any random thick clean sand lenses are encountered. Hole stabilization techniques (such as slurry stabilization) may be required if thick, clean sand lenses are encountered. Stabilization techniques will likely be necessary if excavations penetrate groundwater for perched groundwater. No groundwater was encountered during our field drilling, but perched water may be present if construction takes place with water flowing in the Agua Fria River.

Shallow spread footings bearing upon undisturbed soils below the channel bottom may also be utilized for support of the bridge foundations. Foundation bearing design recommendations for spread footings are presented in the following tabulation:

Alternate Type	*Depth Below Finished-Grade	Footing Width	Allowable Bearing Pressure
Continuous Footings	4 ft.	2-4 ft.	4000 psf
	4 ft.	>4 ft.	4500 psf
	6 ft.	2-4 ft.	5500 psf
	6 ft.	>4 ft.	6000 psf
	8 ft.	2 - 4 ft.	7500 psf
	8 ft.	>4 ft.	8000 psf

*Footings shall bear on natural undisturbed soils at the indicated depth below channel bottom.

The recommended foundation bearing pressures should be considered allowable maximums for dead plus design live loads and may be increased by one-third when considering total loads including transient wind or seismic forces. The weight of the foundation concrete below grade may be neglected in dead load

computations. The minimum recommended footing width is 2 feet for continuous footings.

Estimated settlements for the anticipated loads range from about 1/2 to 3/4 inch. Settlements should occur rapidly as the load is applied, with little or no long term movement.

Lateral Design Parameters: The following tabulation presents recommendations for lateral stability analyses:

[1]Foundation Toe Pressures----------1.33 x allowable
[2]Lateral Backfill Pressures:
Unrestrained walls ---------- 35 psf/ft.
Restrained walls ---------- 60 psf/ft.
Lateral Passive Pressures:
Continuous walls/footings----------250 psf/ft.
Coefficient of Base Friction:
Independent of passive resistance ----------0.40
In conjunction with passive resistance------0.30

[1]Increase in allowable foundation bearing pressure (previously tabulated) for foundation toe pressures due to eccentric or lateral loading. The entire footing bearing surface should remain in compression.

[2]Equivalent fluid pressures for vertical walls and horizontal backfill surfaces. Pressures do not include temporary forces imposed during compaction of the backfill, swelling pressures developed by over-compacted clayey backfill, hydrostatic pressures from inundation of backfill, or traffic or surcharge loads. Walls should be suitably braced during backfilling to prevent damage and excessive deflection.

Compaction of the backfill soils against embedded footings or walls designed to provide passive resistance should be accomplished to a minimum 95 percent of the maximum ASTM D698 density to develop this resistance with low strains. We recommend that compaction against culvert or wing walls within 3 feet of the walls be accomplished using manual compaction equipment only. At least a one foot thickness of freely draining granular material should be provided behind retaining walls. All retaining walls should be provided with through drainage by weep holes to prevent buildup of hydrostatic pressures behind the wall. A filter fabric or graded filter should be provided to reduce migration of fines through drainage weep holes.

Cross Street Pavements: Pavement reconstruction may be required in the vicinity of the bridge crossings at Dysart and El Mirage Road, and for Northern Avenue and Reems Road in the vicinity of the proposed detention basin. Based on the results of subgrade testing in the vicinity of the proposed improvements and computations using the AASHTO design procedure with local modifications, pavement thicknesses were developed for use in these areas. The recommended pavement thicknesses are tabulated below:

Cross Street	Full Depth Asphalt Concrete (in.)	OR	Asphalt Concrete (in.)	Aggregate Base (in.)
Dysart Road	8		5	10
El Mirage Road	6-1/2		4	10*
Reems Road	4-1/2		3	10*
Northern Avenue	6		3*	10*

*Minimum section required by MCDOT Roadway Design Manual, Draft, July 1993.

Pavement materials should not be placed when the subgrade is wet. The surface should be sealed after weathering is apparent to minimize water infiltration directly through the pavement section and retard oxidation. Alternate pavement sections may be developed by exchanging 1 inch of asphalt concrete for 3 inches of aggregate base course.

Detention Basin: Detention basin side slopes for the 160-acre parcel at the northeast corner of Reems Road and Northern Avenue should be stable with side slopes no steeper than 1.5H:1V. However, to satisfy access and landscaping requirements and to reduce erosion, we recommend slopes no steeper than 6H:1V. Shallow lined nuisance flow channels up to 3 feet deep should be stable with side slopes no steeper than 1H:1V.

Percolation tests were conducted in the area of the proposed retention basin, and are reported in Appendix A. These tests were conducted in uncased test borings and include the effects of side area. Accurate modeling of large scale vertical percolation is not possible in small diameter test borings, which can be expected to over estimate the percolation rate. Additionally, percolation rates will be further

reduced over time by the accumulation of fine sediment at the bottom of the detention basin. The clay soils in the vicinity of the detention basin should be expected to have slow to moderately slow percolation rates. Near surface percolation rates should be significantly slower due to the clay surface soils, and because shallow surface ponds (as from nuisance flows) will have lower heads than the basin.

For preparation of earthwork plans, a shrinkage factor from in-place to new fills for the upper tilled soils (upper 1.5 - 2 feet) in the proposed detention basin excavation area of 20% should be used. Lower undisturbed materials should experience about 12% shrinkage. We estimate about 1 inch of shrinkage of the scarified and recompacted zone under new fills, and about 1 inch of settlement under a 12 foot fill height over time. In addition, we estimate the fill itself will settle about 1.5% of its height.

Excavation Conditions: The test drilling and field sampling at the site were performed for design purposes. It is not possible to accurately correlate auger drilling results with the ease or difficulty of digging for various types and sizes of excavation equipment. We present the following general comments regarding excavatability for the designers' information with the understanding that they are approximations based only on test boring data. More accurate information regarding excavatability should be evaluated by contractors or other interested parties from test excavations using the intended equipment.

Shallow excavations into the clayey site soils should be possible with conventional equipment. Below a depth of 10 to 15 feet, calcareous cementation of up to moderate intensity was noted in many test borings. Excavation into these deeper materials could be more difficult, especially if heavily cemented zones are encountered, and may require the use of specialized equipment in some areas. Excavations should be braced or sloped as required to provide personnel safety and satisfy local safety code regulations.

Construction Excavations: The following criteria are presented for temporary slopes for utility placement and bridge construction.

1. Unbraced temporary slopes in the existing clay soils should stand at slopes of 1H:1V. Locally, it may be necessary to further flatten slopes if very clean, loose sand lenses of significant thickness or utility backfills are encountered. Temporary slopes of 1.5H:1V should stand in the sandy soils. As an alternative, localized bracing or shoring may be required in areas of caving.

2. Surface areas behind the crest of excavations should be graded so that surface waters do not pond within 15 feet of the crest, nor drain into the excavation from roadway drainage.

3. Heavy material stockpiles should not be placed within 20 feet of the crest of slopes. Similarly, heavy construction equipment should not pass over or be parked within 20 feet of the crest.

4. The crest of slopes should be monitored daily for evidence of movement or potential problems.

The design of any bracing systems should be reviewed by the geotechnical engineer. Also, observations should be made by the geotechnical engineer during excavating to evaluate site conditions and determine if modifications are necessary in excavation procedures. If unbraced slopes are utilized, some surface raveling, erosion, and spalling should be expected unless measures are taken to stabilize exposed cut surfaces.

Corrosion: Corrosion is most likely to occur in fills and natural soils with high moisture contents. Laboratory tests were performed on selected samples to determine pH, minimum resistivity, and soluble salts and sulfates. The results are presented in Appendix B. Based on the test values, the corrosion potential to concrete is negligible based on current testing. (The results from the Morton Salt Bridge test borings (B-13 and B-15) are still pending). Therefore, concrete in contact with soils should use Type II cement. A moderate high potential for corrosion of buried metal conduits is indicated in areas where soil moisture contents are high. Metal culverts should be either aluminum alloy or bituminous coated. Consideration should be given to the use of approved, non-metallic or cathodic protected conduits for underground utilities. Also, special protection may

all others are West of Dysart

No Al. pipe where to be spliced to steel

be necessary where dissimilar metals are placed in close proximity or are joined. We recommend that these test results be reviewed by a person or firm experienced in corrosion potential.

Structural Backfills: Backfill behind subsurface walls designed to support utilities, pavement, or other facilities should be compacted to density criteria presented under Site Grading. Backfills should consist of granular soils which exhibit low expansive potentials. On-site soils may be used in structural backfills more than 3 feet below the surface. High plasticity on-site soils may not be used in structural backfills. If backfills are not compacted as recommended, subsidence may result in areas adjoining backfilled subsurface walls. Even properly compacted deep backfills may tend to settle differentially relative to subsurface walls and should not be used for support of adjoining facilities or utilities prone to damage from differential settlements.

Saturation of backfill and development of hydrostatic pressures is possible in below-grade areas due to breakage of utility lines embedded in loose backfills, infiltration of surface water through loose backfills, or leakage from the proposed channel. Backfill compaction should be accomplished by mechanical methods, with water settling or jetting not permitted.

Site Soil Workability: Below the culvert bottom the moisture content of existing site soils should be maintained between optimum and optimum +3 percent (ASTM D698) during and subsequent to site grading to reduce expansive potentials. At these conditions, some pumping may be experienced under dynamic loading if the compaction is done by very heavy equipment (i.e., loaded scrapers, water-pulls, etc.). We would not consider some pumping detrimental in areas below the culvert bottom (i.e., static loading conditions) provided specified densities are obtained. Lighter compaction equipment and/or drying of wet soils may be used to reduce pumping if this condition becomes severe.

In bituminous paved areas, the moisture content of the subgrade and backfill should be maintained at 2 percent below optimum or lower during site grading to reduce the potential for pumping. If in-situ moisture contents are higher than this at the time of construction, pumping may occur, and special precautions should be taken to prevent disturbance, equipment mobility problems, and loss of shear

strength in the subgrade. These precautions may include spreading and drying of wet soils, removal and replacement of wet soils, construction of temporary gravel roads at channelized traffic areas, and/or use of lighter compaction equipment.

PART II

MATERIALS

FILL MATERIALS

All fill materials should be soils free of vegetation, debris, organic contaminants, and fragments larger than *6 inches in size. The existing site surface soils may be used for fill or backfill more than 3 feet below final grade when compacted as described under site grading. High plasticity site soils should not be used in any structural fills or backfills.

Imported soil used for fills below box culverts or backfills around box culverts or under pavements should be granular soils conforming to the following specification requirements:

Maximum particle size ..... 6 inches*
Maximum percent expansion 1.5**

* Maximum size may be reduced at engineer's direction to satisfy trenching and landscaping requirements, etc.

** Performed on sample remolded to 95 percent of the maximum ASTM D698 density and 2 percent below optimum moisture under a 100 psf surcharge pressure.

Imported soil used for backfill of over-excavated high plasticity clays below the channel invert should be clayey sand soils conforming to the following specification requirements:

Maximum particle size ..... 4 inches*
Maximum percent expansion 1.5*
Percent Passing No. 200 sieve.....15 to 50
Plasticity Index.....5 to 15

* Performed on sample remolded to 95 percent of the maximum ASTM D698 density and 2 percent below optimum moisture under a 100 psf surcharge pressure.

Imported soil for use in backfilling in overexcavated earth fissure areas should be clayey soils conforming to the following specification requirements:

Maximum Plasticity Index 12
Minimum Percent Passing No. 200 Sieve 1.5*

BASE COURSE

Base course materials for use beneath interior floor slabs and pavements should be well graded sand and gravel materials meeting the Maricopa Association of Governments' Specifications for Aggregate Base Course Materials (Section 702).

PAVEMENT

Pavement materials should be in accordance with the requirements of the Maricopa Association of Governments Standard Specifications for Asphalt Concrete (Section 710, Type C-3/4).

PART III

EXECUTION

SITE GRADING

The following recommendations are provided for the grading and excavation required for the proposed channel improvements. All phases of earthwork should be performed under observation and testing directed by the geotechnical engineer.

1. Excavate as necessary for placement of the new channel lining and box culverts. The base of the excavations for proposed box culverts should be flat and should extend at least 3 feet beyond the culvert walls. Sides should be braced or sloped in accordance with previous recommendations under Construction Excavations.

2. The base should be cleaned of all organic contaminants, debris, former facility remnants and any loose or disturbed soils encountered. The cleaned surface should be observed for evidence of debris-laden soils, disturbance, water-softened soils (as in existing shallow unlined channel areas), concealed facility remnants, high plasticity clays, or loose zones requiring additional removal. The identification of the high plasticity clays will probably require visual inspection by a geotechnical engineer. Where high plasticity clays are present, they should be removed from at least within two feet of the channel lining. The surface should then be scarified, moisture conditioned, and compacted to a minimum 8 inch thickness. A 4-inch thick layer of aggregate base may be used as a construction platform or leveling course under the box culverts and channel bottom, if desired.

   conting. item?

3. Place backfill as required to elevate site areas to required grade or replace disturbed or removed soil. Fill material should be placed and compacted in horizontal lifts compatible with the compaction equipment being used.

4. Compaction of exposed site soil, backfill, fill, and base course materials should be accomplished to the following density criteria.

Material	Percent Compaction (ASTM D698)
Subgrade Soil:	
Below box culverts	95 min.
Below base of footings	95 min.
Below pavement	95 min.

Material	Percent Compaction (ASTM D698)
Backfill:	
Below channel lining	95 min.
Aggregate Base Course:	
Below channel lining	95 min.
Below pavement	100 min.
Fill:	
In proposed golf course area	90 min.

Compaction of on site soils in scarified zones or in new fills more than 3 feet below final grade should be accomplished at a moisture content between optimum and optimum +3%. Compaction of granular imported soil below the channel lining, the box culverts or footings should be accomplished at a moisture content between optimum ±3%. Compaction of imported clayey sand soil backfill in areas of high plasticity clay overexcavation should be accomplished at a moisture content between optimum and optimum +3%. Compaction of exposed soil and fill material within 3 feet of asphalt pavement should be accomplished at a moisture content 2 % below optimum or lower.

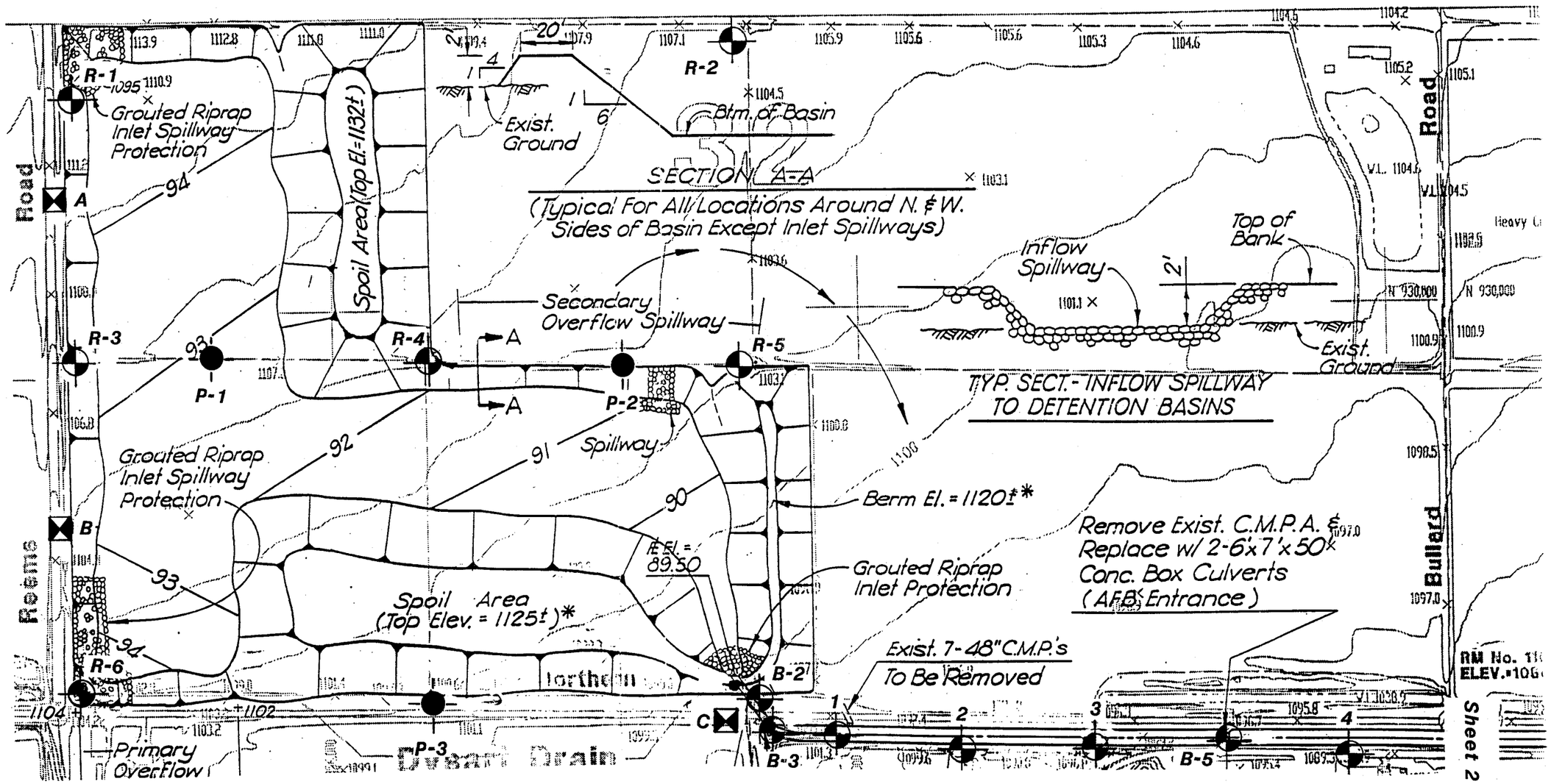
On site undisturbed soils or compacted soils subsequently disturbed or removed by construction operations should be replaced by materials compacted as specified above.

PAVING

Placement requirements for paving should be in accordance with the Maricopa Association of Governments' Specifications for Asphalt Concrete Pavement (Section 321). Observation and testing should be performed as necessary to verify conformance with these recommended specifications, especially compaction requirements for asphaltic concrete surfacing.

APPENDIX A

FIELD RESULTS



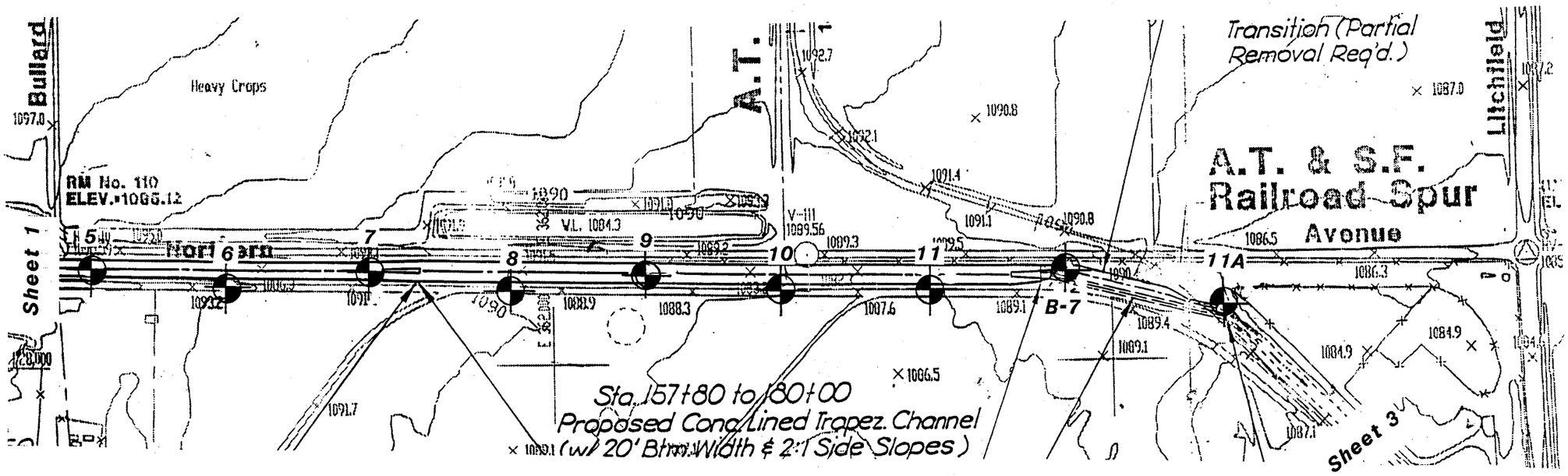
LEGEND

- Test Boring Location
- Subgrade Sample Location
- Percolation Test Location

N

SITE PLAN
Sheet 1 of 4
Project No. 93-1033

THOMAS-HARTIG & ASSOCIATES, INC.



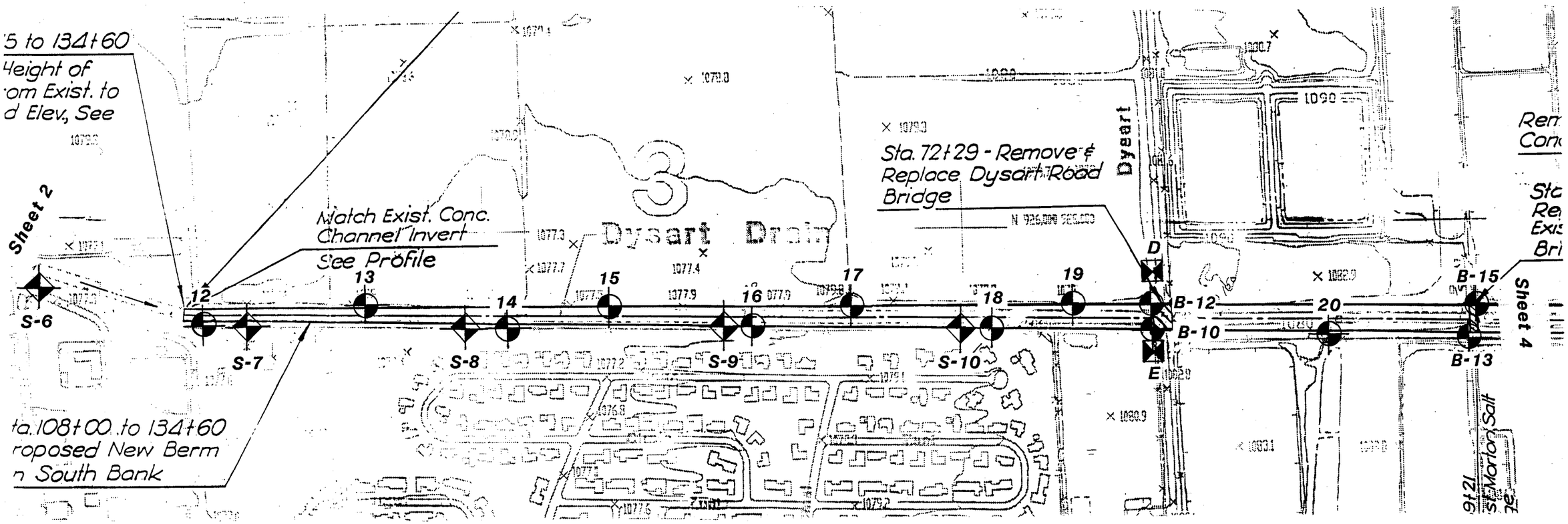
LEGEND

Test Boring Location

N

SITE PLAN
Sheet 2 of 4
Project No. 93-1033

THOMAS-HARTIG & ASSOCIATES, INC.



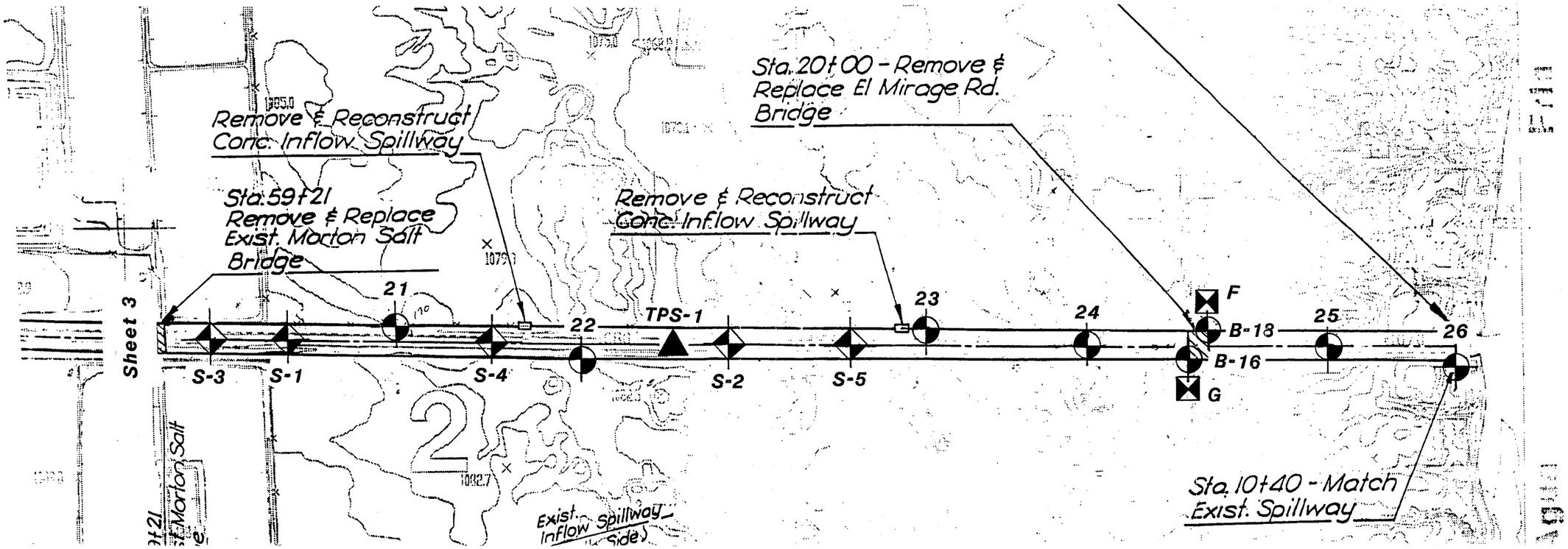
LEGEND

- Test Boring Location
- Speedie & Associates Test Boring
- Subgrade Sample Location

N

SITE PLAN
Sheet 3 of 4
Project No. 93-1033

THOMAS-HARTIG & ASSOCIATES, INC.



LEGEND

- Test Boring Location
- Speedie & Associates Test Boring
- Speedie & Associates Test Pit
- Subgrade Sample Location

N

SITE PLAN
Sheet 4 of 4
Project No. 93-1033

THOMAS-HARTIG & ASSOCIATES, INC.

LEGEND

SOIL CLASSIFICATION

COARSE-GRAINED SOIL

More than 50% larger than 200 sieve size

SYMBOL	LETTER	DESCRIPTION	MAJOR DIVISIONS
	GW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES. LESS THAN 5% - #200 FINES	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size
	GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES. LESS THAN 5% - #200 FINES	
	GM	SILTY GRAVELS. GRAVEL-SAND-SILT MIXTURES. MORE THAN 12% - #200 FINES	
	GC	CLAYEY GRAVELS. GRAVEL-SAND-CLAY MIXTURES. MORE THAN 12% - #200 FINES	
	SW	WELL-GRADED SANDS OR GRAVELLY SANDS. LESS THAN 5% - #200 FINES	SANDS More than half of coarse fraction is smaller than No. 4 sieve size
	SP	POORLY-GRADED SANDS OR GRAVELLY SANDS. LESS THAN 5% - #200 FINES	
	SM	SILTY SANDS. SAND-SILT MIXTURES MORE THAN 12% - #200 FINES	
	SC	CLAYEY SANDS. SAND-CLAY MIXTURES MORE THAN 12% - #200 FINES	

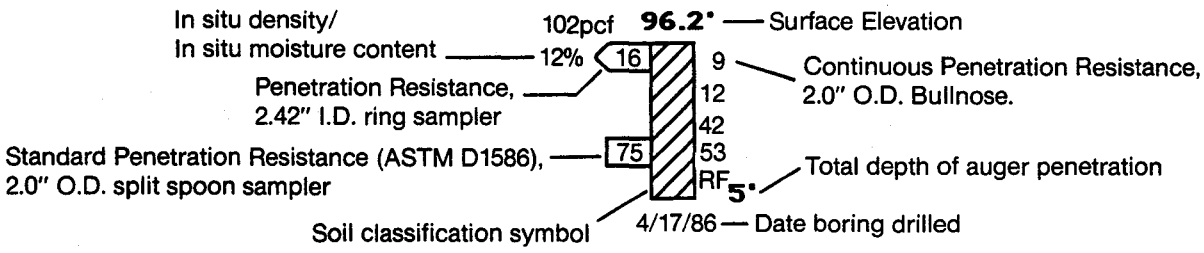
FINE-GRAINED SOIL

More than 50% smaller than 200 sieve size

SYMBOL	LETTER	DESCRIPTION	MAJOR DIVISIONS
	ML	INORGANIC SILTS. ROCK FLOUR. AND FINE SANDY OR CLAYEY SILTS OF LOW TO MEDIUM PLASTICITY	SILTS AND CLAYS Liquid limit less than 50
	CL	INORGANIC CLAYS. GRAVELLY CLAYS. SANDY CLAYS. SILTY CLAYS. AND LEAN CLAYS OF LOW TO MEDIUM PLASTICITY	
	OL	ORGANIC SILTS AND ORGANIC SILT-CLAY MIXTURES OF LOW TO MEDIUM PLASTICITY	
	MH	INORGANIC SILTS. MICACEOUS OR DIATOMACEOUS. AND FINE SANDY OR CLAYEY SILTS OF HIGH PLASTICITY	SILTS AND CLAYS Liquid limit greater than 50
	CH	INORGANIC CLAYS. FAT CLAYS. AND SILTY CLAYS OF HIGH PLASTICITY	
	OH	ORGANIC CLAYS AND ORGANIC SILTS OF MEDIUM TO HIGH PLASTICITY	
	PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	

LEGEND FOR GRAPHICAL BORING LOGS:

Log denotes visual approximation unless accompanied by mechanical analysis and Atterberg limits.



PENETRATION RESISTANCE: Blows per foot using 140 lb. hammer with 30" free-fall unless otherwise noted.

GRAIN SIZES							
U.S. STANDARD SERIES SIEVE				CLEAR SQUARE SIEVE OPENINGS			
200	40	10	4	3/4"		3"	12"
SILTS & CLAYS DISTINGUISHED ON BASIS OF PLASTICITY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

MOISTURE CONDITION (INCREASING MOISTURE →)

DRY	SLIGHTLY DAMP	DAMP (Plastic Limit)	MOIST	VERY MOIST	WET (SATURATED) (Liquid Limit)

CONSISTENCY CORRELATION		RELATIVE DENSITY CORRELATION	
CLAYS & SILTS	BLOWS/FOOT*	SANDS & GRAVELS	BLOWS/FOOT*
VERY SOFT	0-2	VERY LOOSE	0-4
SOFT	2-4	LOOSE	4-10
FIRM	4-8	MEDIUM DENSE	10-30
STIFF	8-16	DENSE	30-50
VERY STIFF	16-32	VERY DENSE	OVER 50
HARD	OVER 32		

*Number of blows of 140 lb. hammer falling 30" to drive a 2" O.D. (1-3/8" I.D.) split-spoon sampler (ASTM D1586).

Project No. 93-1033

THOMAS-HARTIG & ASSOCIATES, INC.

LEGEND OF SOIL TYPES

SANDY CLAY (CL, SC-CL); light brown to brown; firm to very stiff; low to medium plasticity; stratified; variable quantities primarily fine to medium sand; variable light to moderate calcareous cementation; damp to moist.

CLAY (CH, CL-CH); light brown to dark brown; stiff to hard; medium to high plasticity; stratified; traces to some fine sand; uncemented to moderate calcareous cementation; slightly damp to moist.

SILTY SAND (SM, SM-ML); light brown; medium dense to dense; subangular to subrounded primarily fine to coarse sand with low plasticity fines and occasional lenses with fine gravel; uncemented to light calcareous cementation; damp to moist.

SANDY SILT (MH); light greenish brown; stiff to very stiff; medium plasticity; light calcareous cementation; damp.

SILTY SAND AND GRAVEL (SM-GM); light brown; dense to very dense; angular to subrounded fine to coarse sand and fine to medium gravel; non-plastic to low plasticity fines; damp to moist.

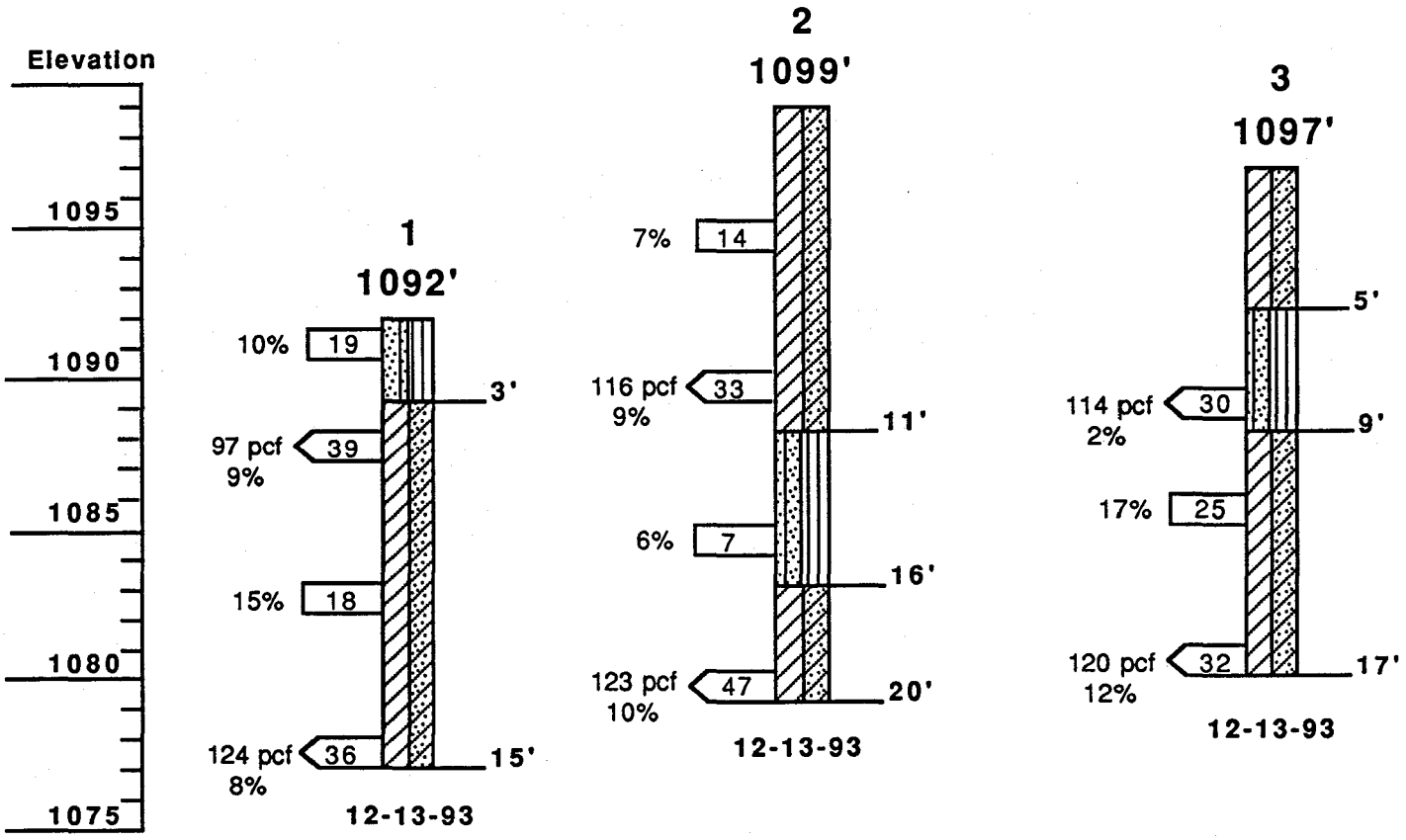
CLAYEY SAND (SC); brown; medium dense to dense; fine to coarse subangular to subrounded fine to coarse sand and traces to some gravel with low plasticity fines; damp to moist.

SAND (SP); light brown; medium dense; fine to coarse subrounded to rounded fine to coarse sand with traces to some fine gravel; moist.

SAND AND GRAVEL (GW, GW-GM); light brown; medium dense to very dense; angular to subangular fine sand to medium gravel with subangular to subrounded medium gravel to cobbles; damp to moist.

SPRAY-ON SHOTCRETE CHANNEL LINING (see individual logs for thicknesses).

GRAPHICAL BORING LOGS



Elevation approximate from Preliminary Design Profile by the WLB Group

No free groundwater was encountered in any of the borings during drilling.

All borings drilled with 4" diameter continuous flight auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

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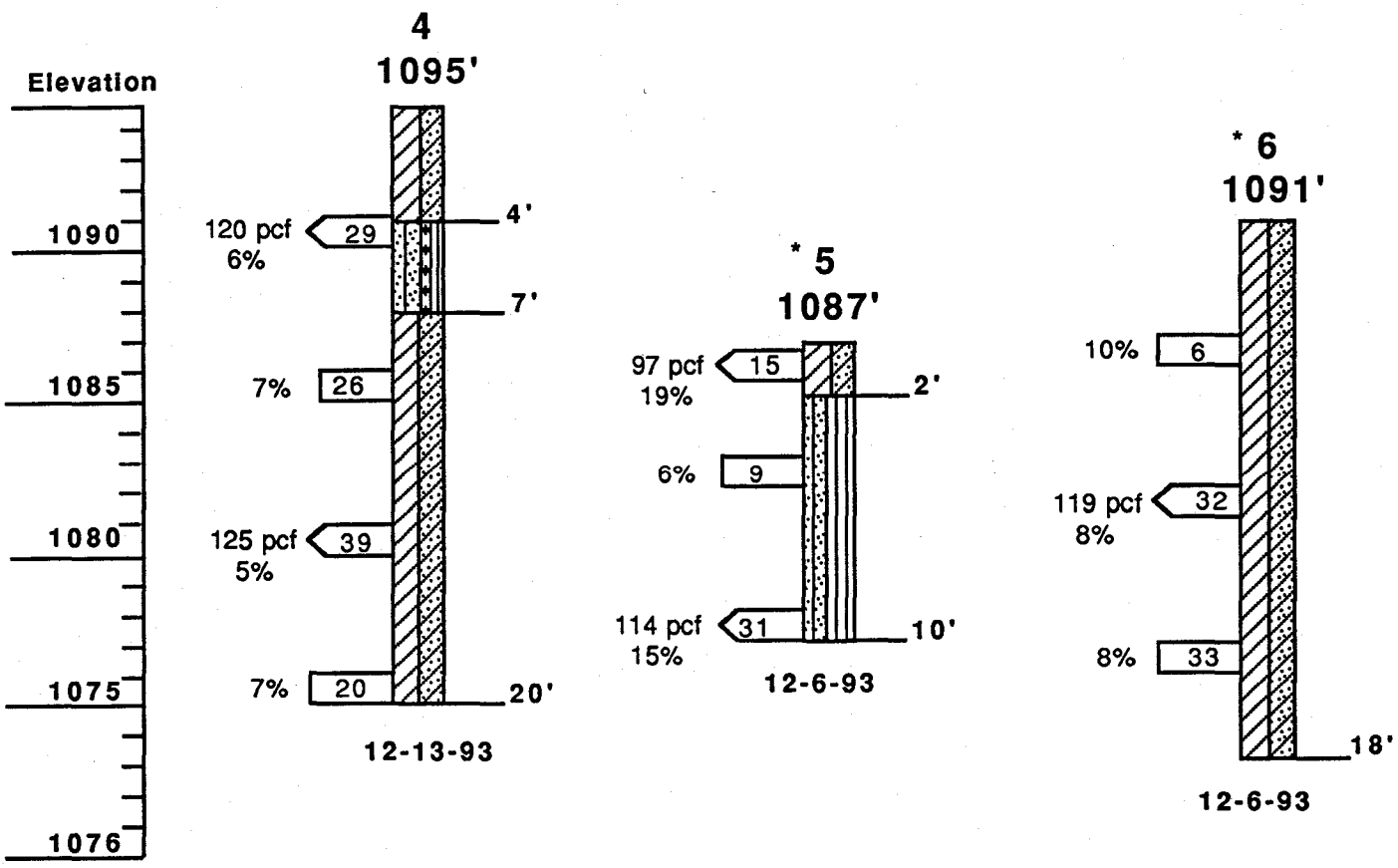

Elevation approximate from Preliminary Design Profile by the WLB Group

* Drilled with a 7" diameter hollow stem auger

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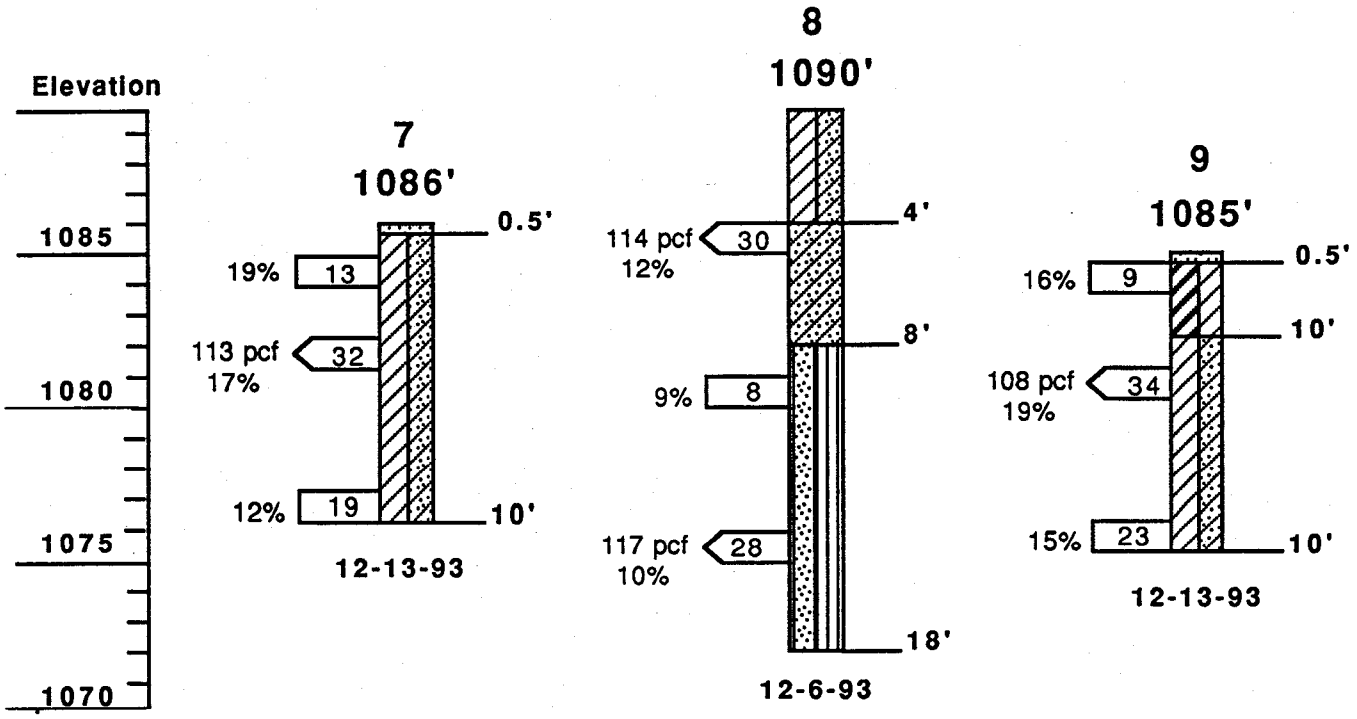

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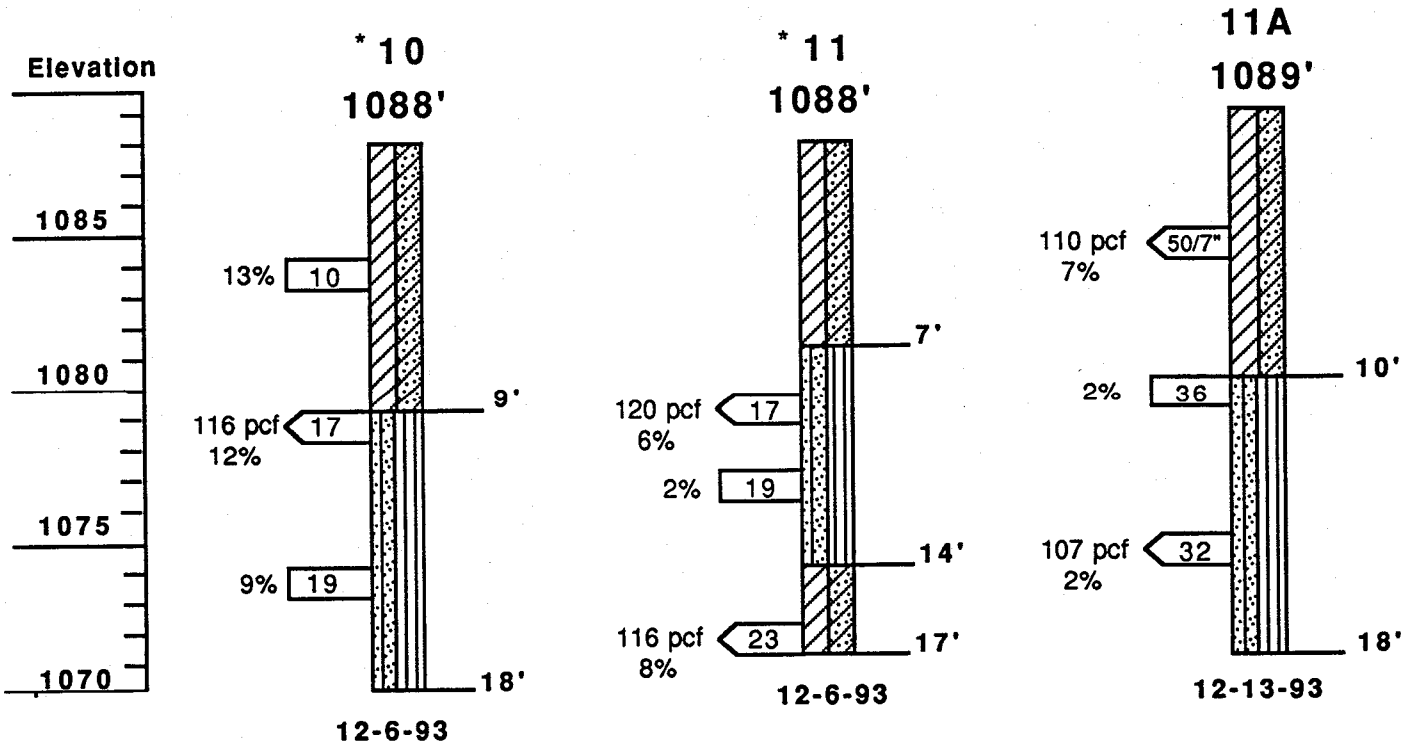

* Drilled with a 7" diameter hollow stem auger

Elevation approximate from Preliminary Design Profile by the WLB Group

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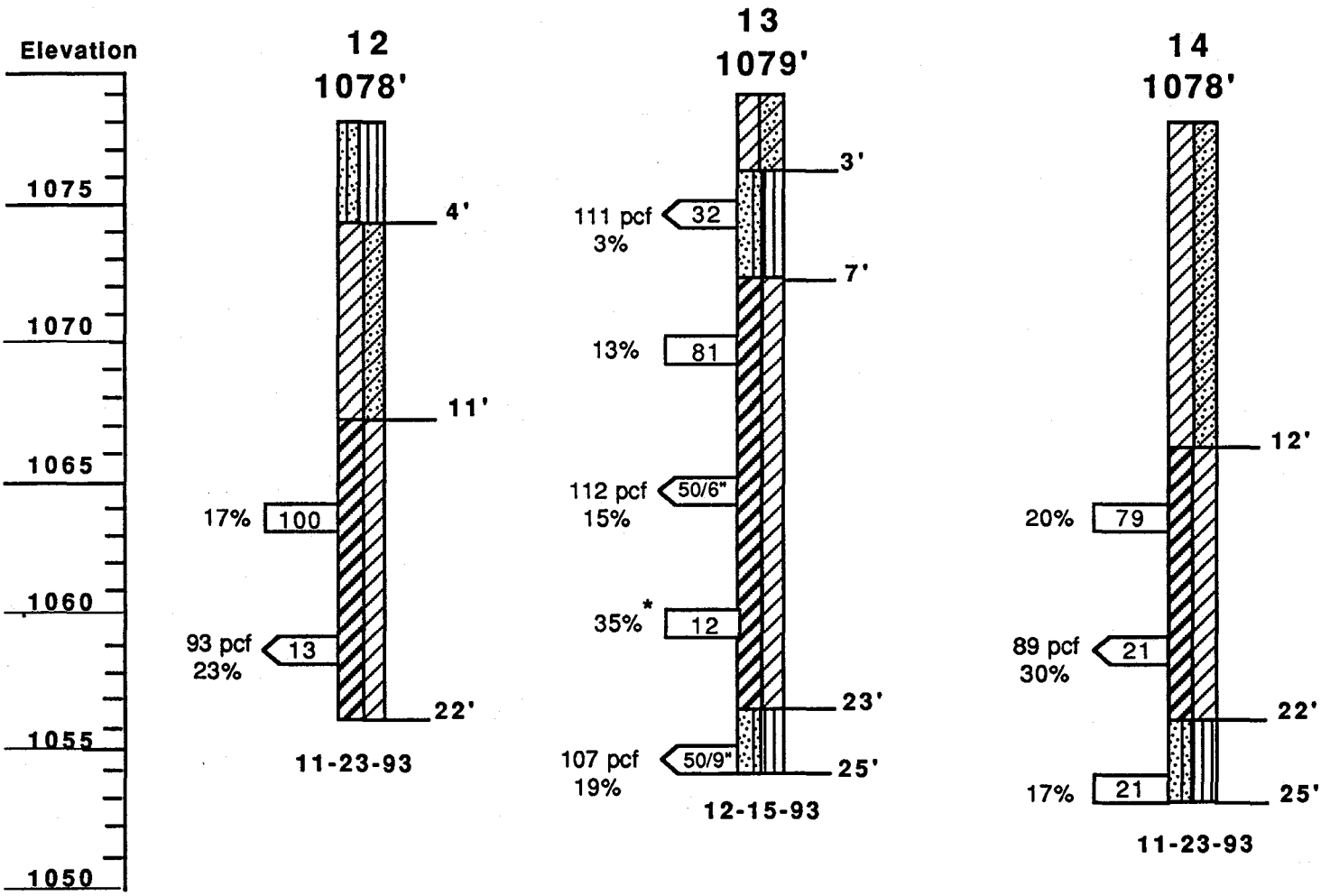

* Water added to facilitate drilling

Elevation approximate from Preliminary Design Profile by the WLB Group

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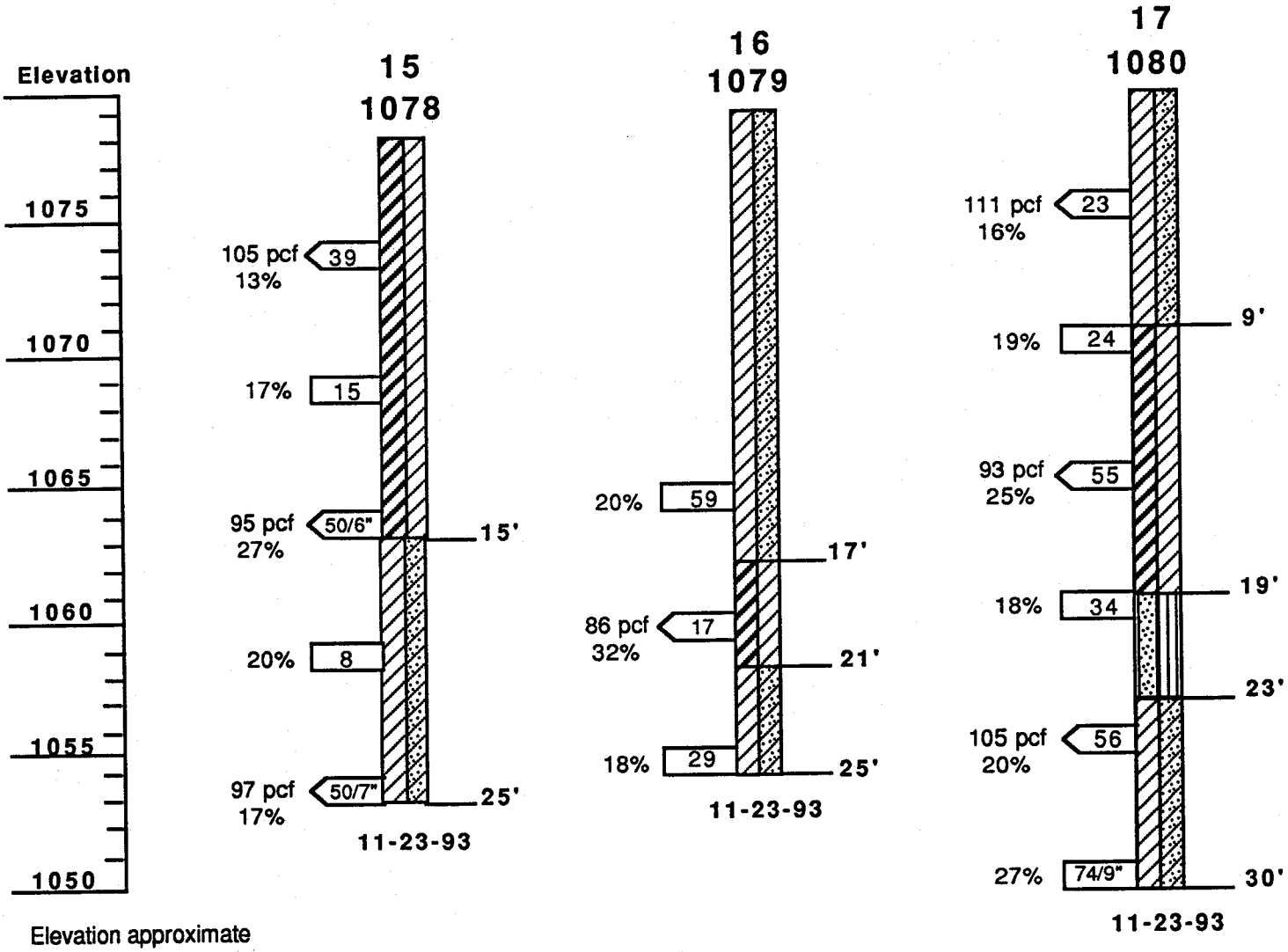

Elevation approximate from Preliminary Design Profile by the WLB Group

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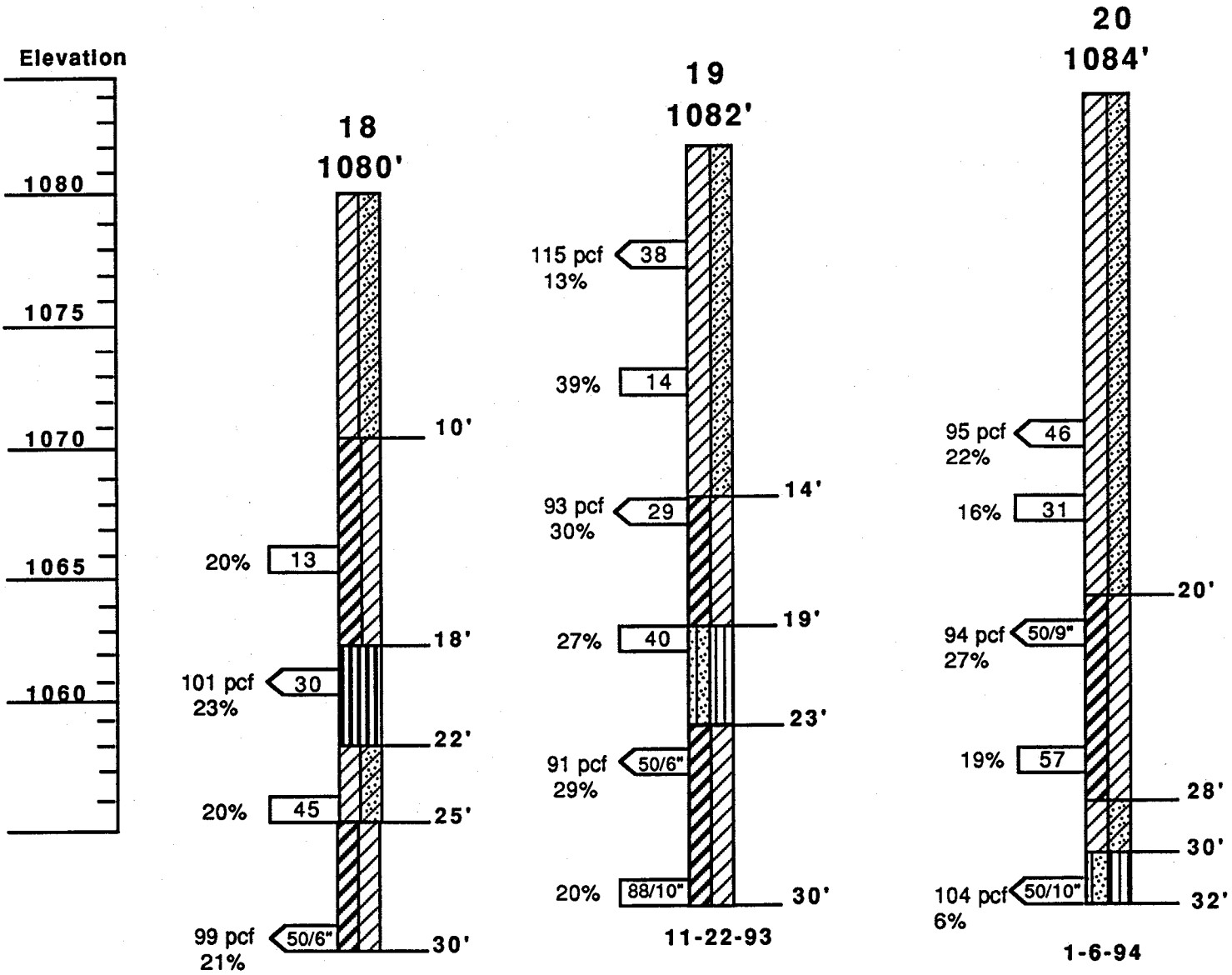

Elevation approximate
from Preliminary Design
Profile by the WLB Group

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All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

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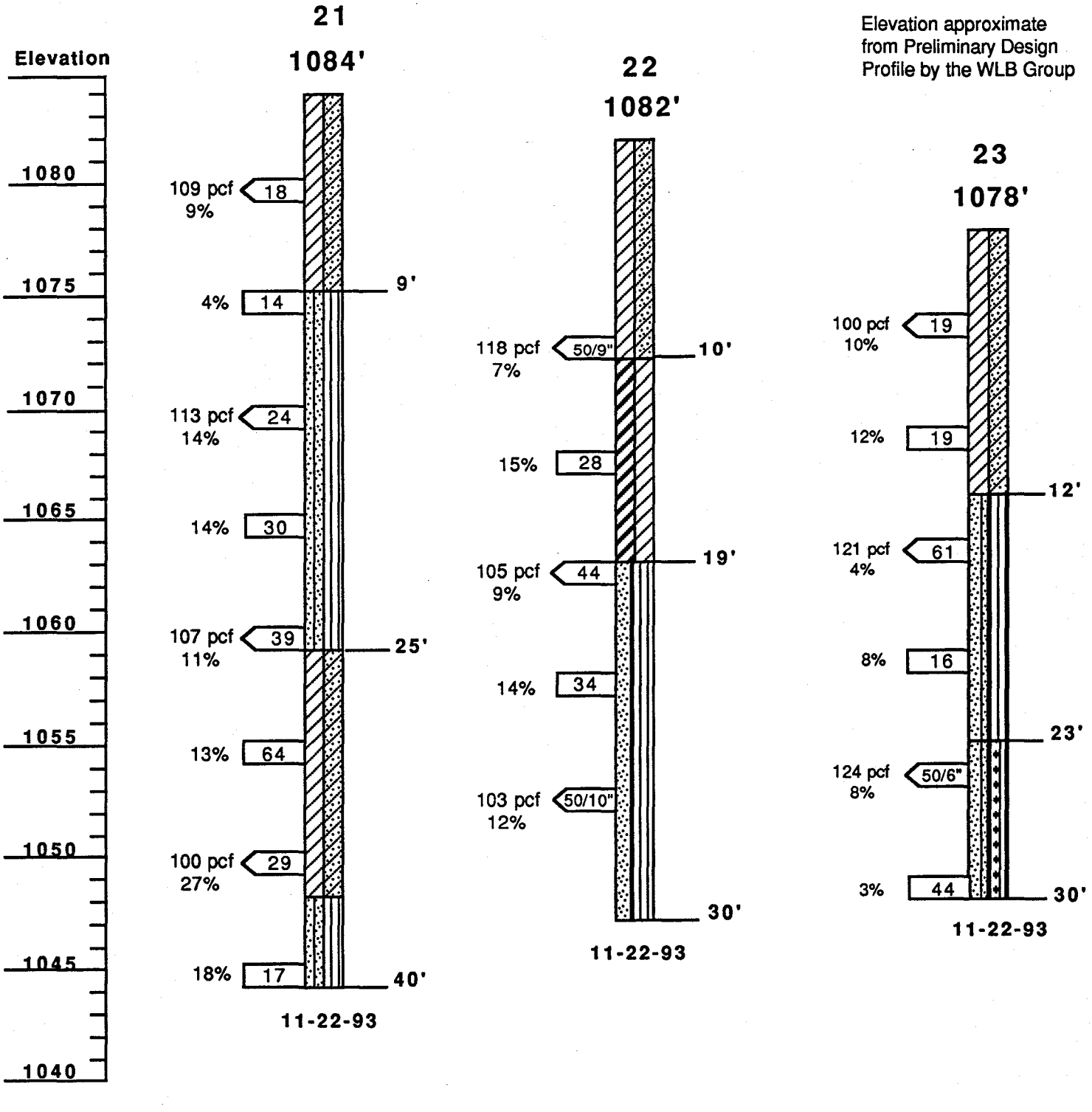

No free groundwater was encountered in any of the borings during drilling.

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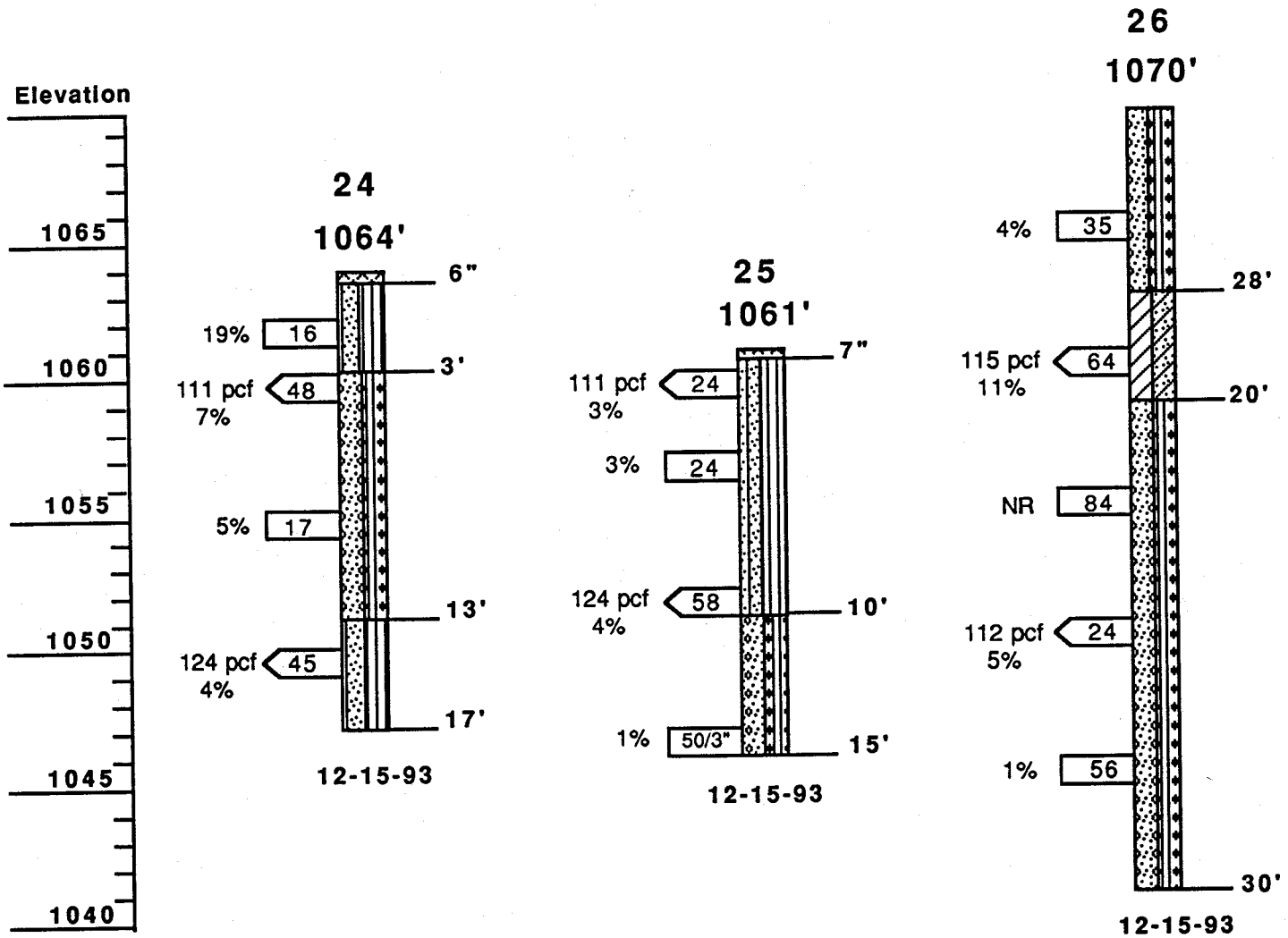

Elevation approximate from Preliminary Design Profile by the WLB Group

NR = No Recovery

No free groundwater was encountered in any of the borings during drilling.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

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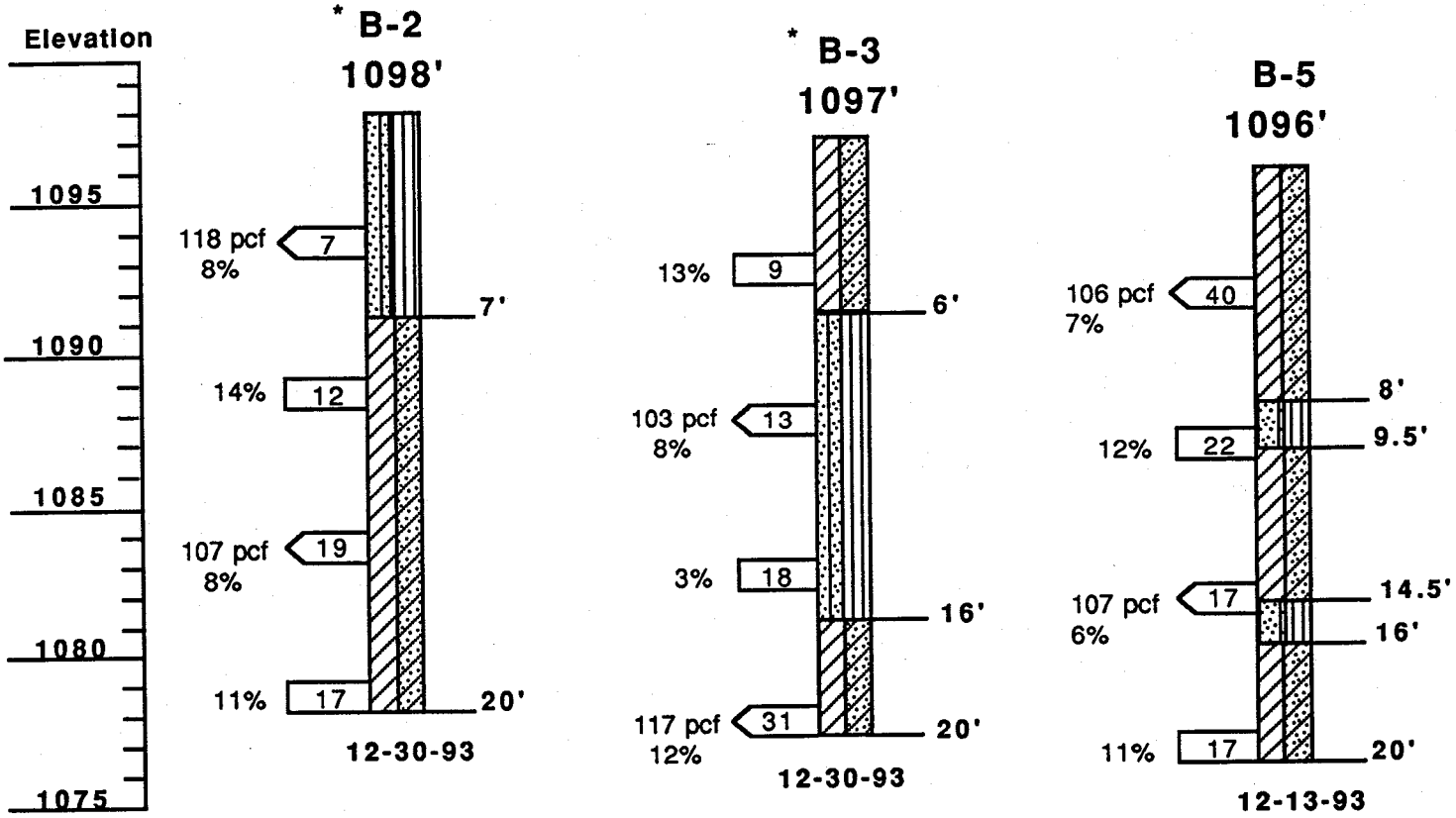

* Drilled with a 7" diameter hollow stem auger

Note: B1, B4, and B6 not drilled

Elevation approximate from Preliminary Design Profile by the WLB Group

No free groundwater was encountered in any of the borings during drilling.

All borings drilled with 4" diameter continuous flight auger unless otherwise noted.

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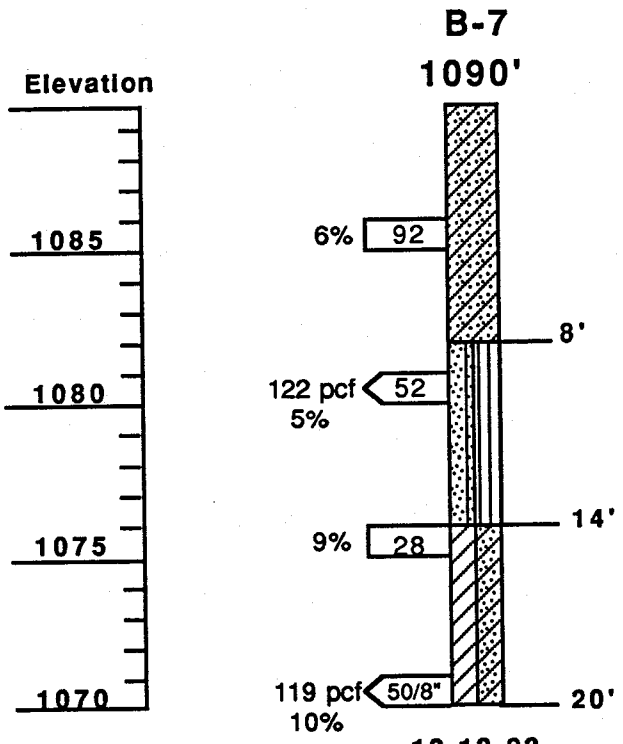

Note: B8 and B9 not drilled

Elevation approximate from Preliminary Design Profile by the WLB Group

No free groundwater was encountered in any of the borings during drilling.

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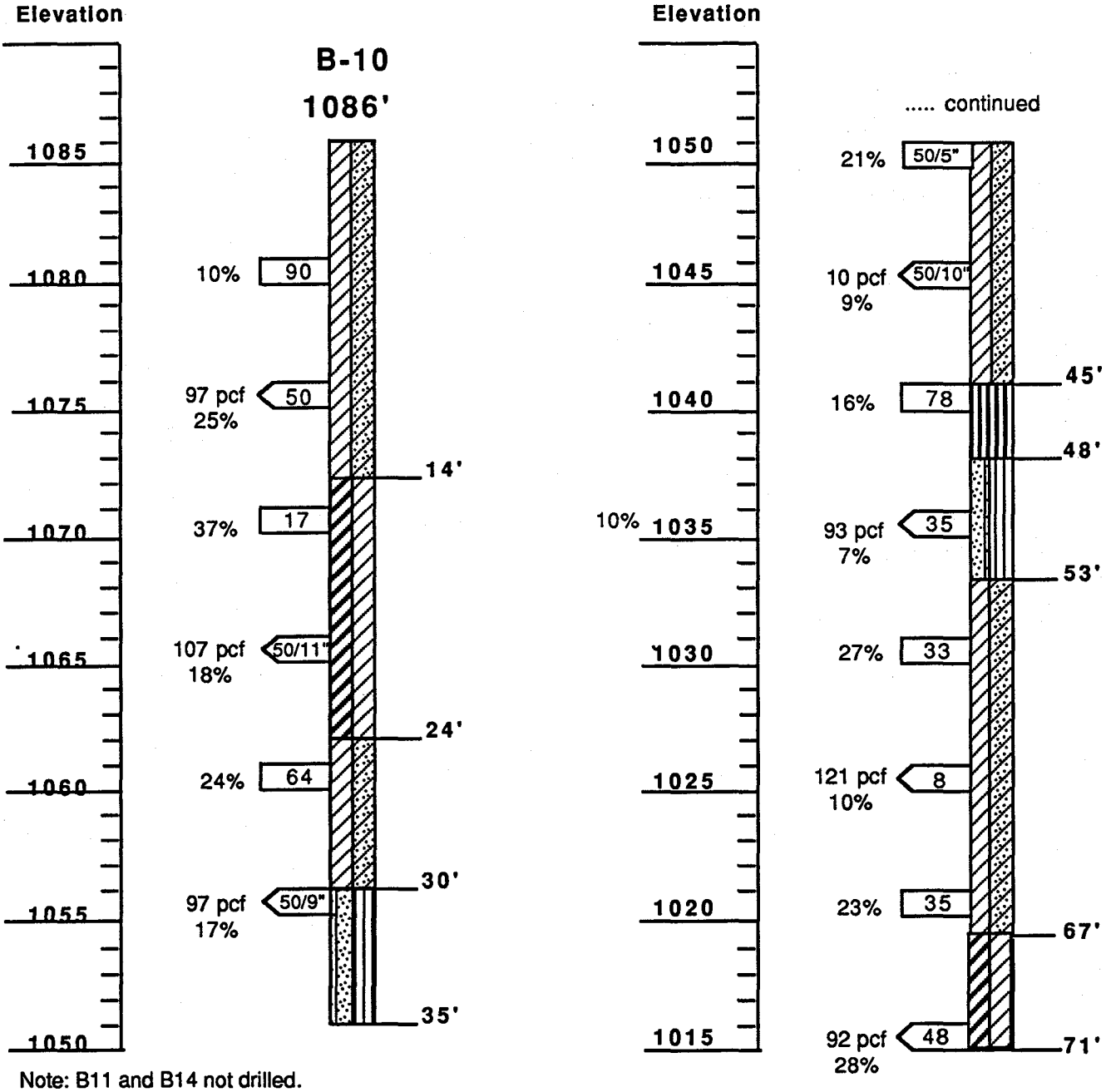
GRAPHICAL BORING LOGS

Elevation approximate from
Preliminary Design Profile
by the WLB Group



Note: B11 and B14 not drilled.
B12, B13, B15 to be added on
completion of testing.

No free groundwater was encountered in any of the borings during drilling.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

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GRAPHICAL BORING LOGS

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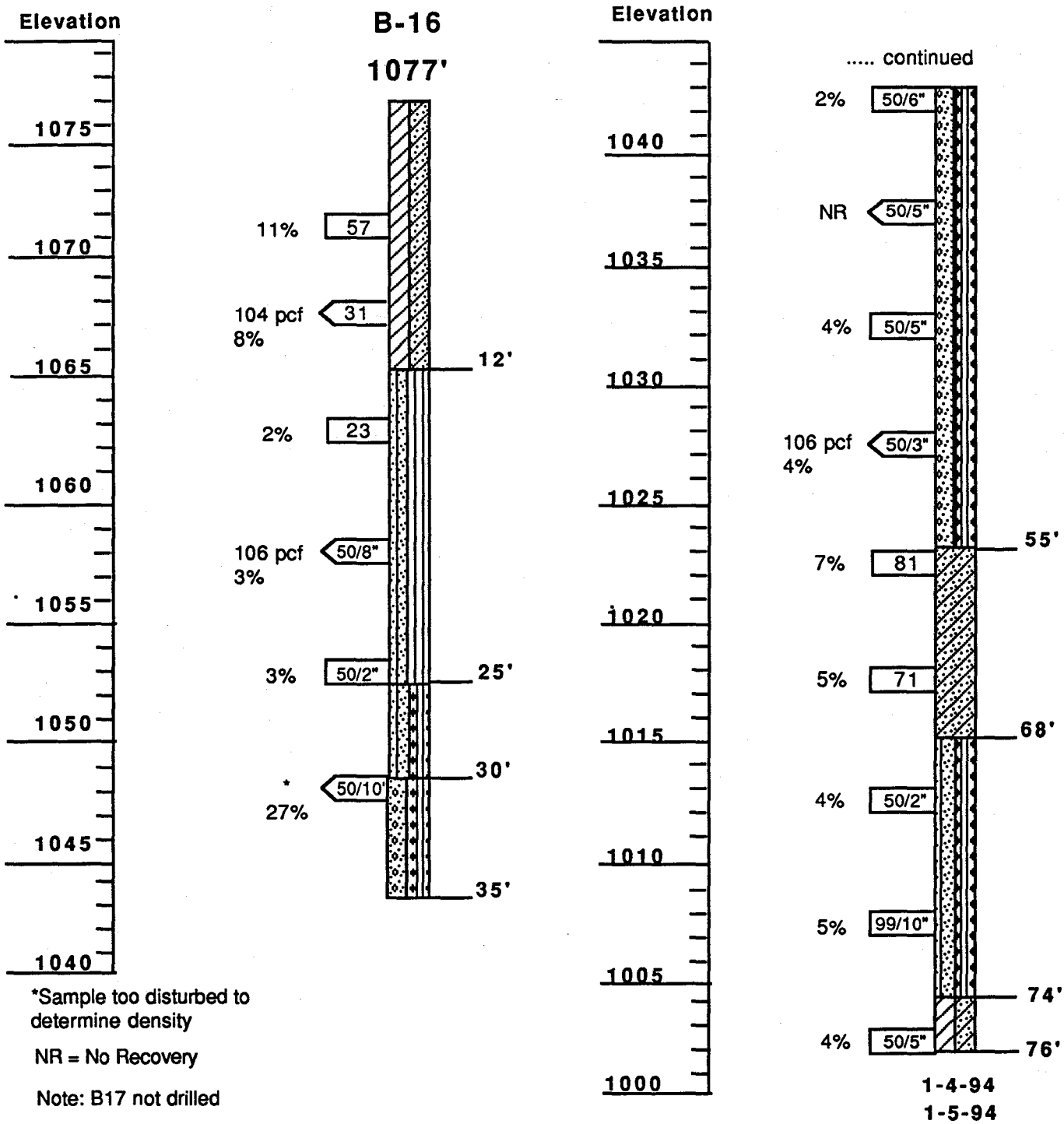
Design Profile by the WLB Group



*Sample too disturbed to determine density

NR = No Recovery

Note: B17 not drilled

No free groundwater was encountered in any of the borings during drilling.

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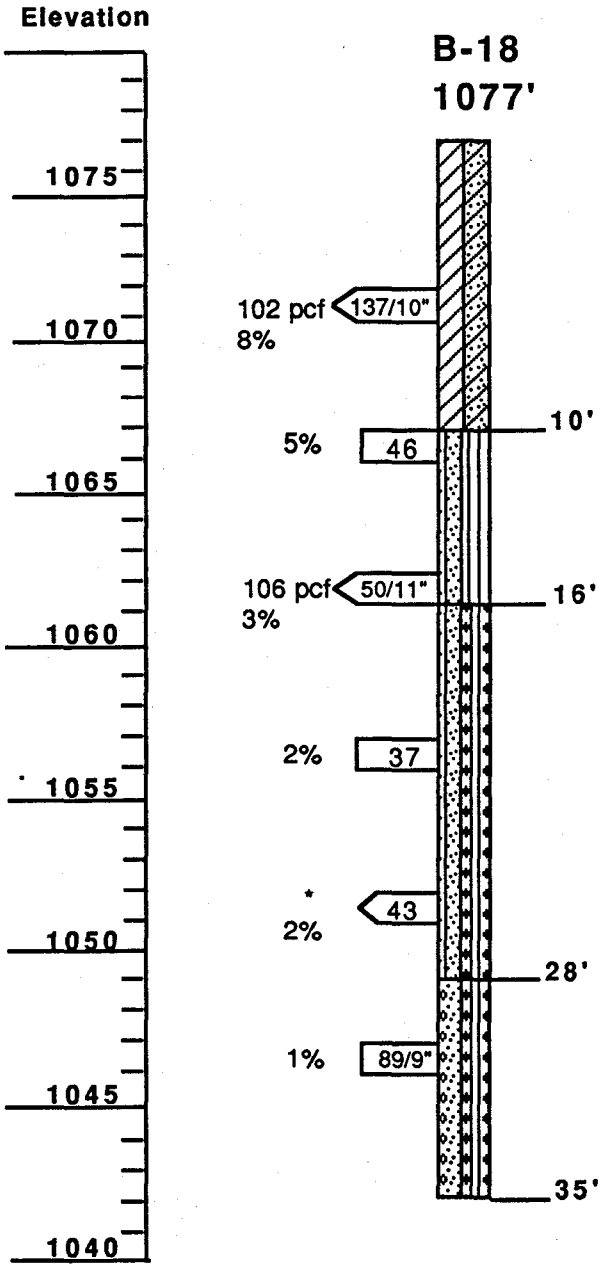

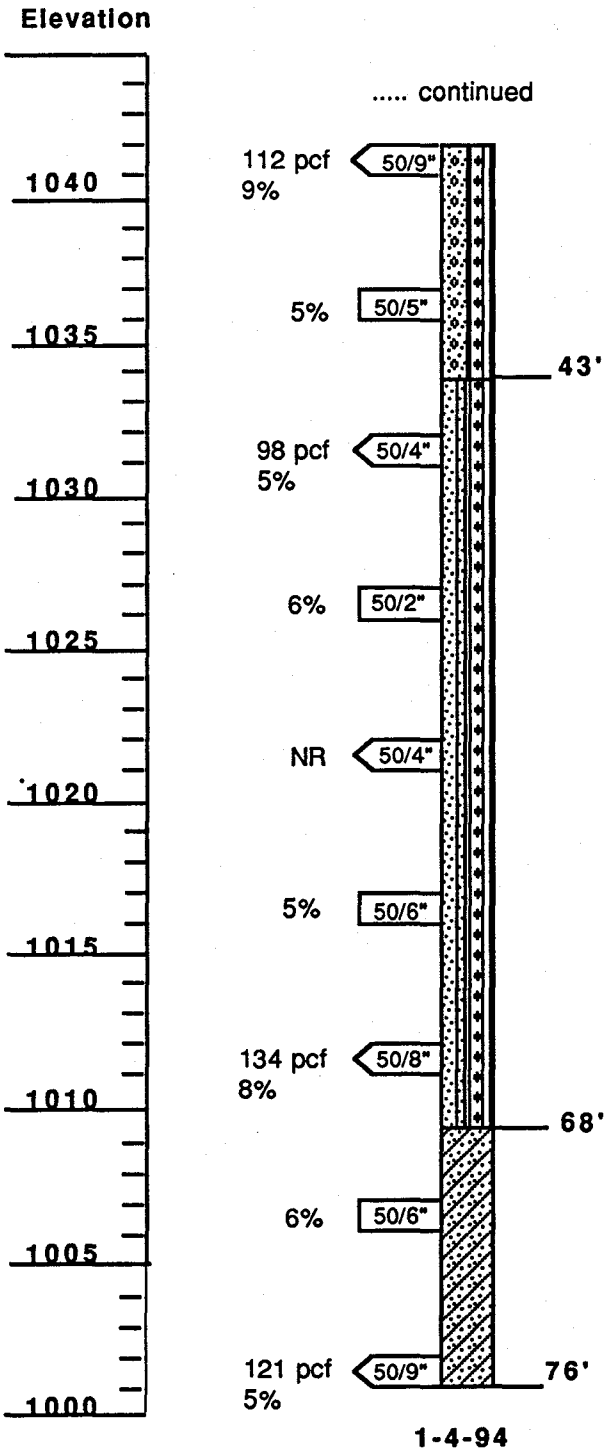

*Sample too disturbed to determine density

NR = No Recovery

No free groundwater was encountered in any of the borings during drilling.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

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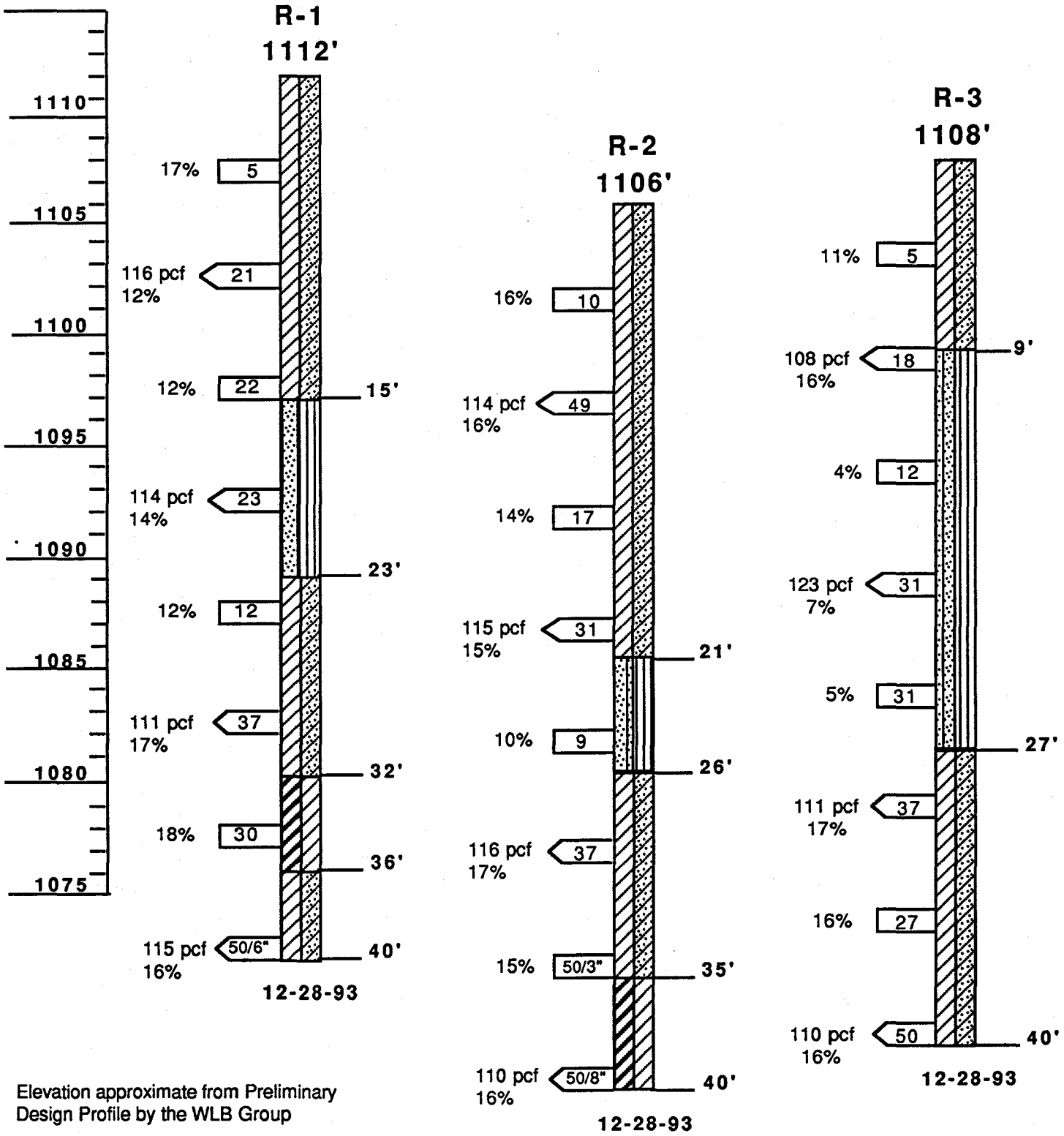

Elevation approximate from Preliminary Design Profile by the WLB Group

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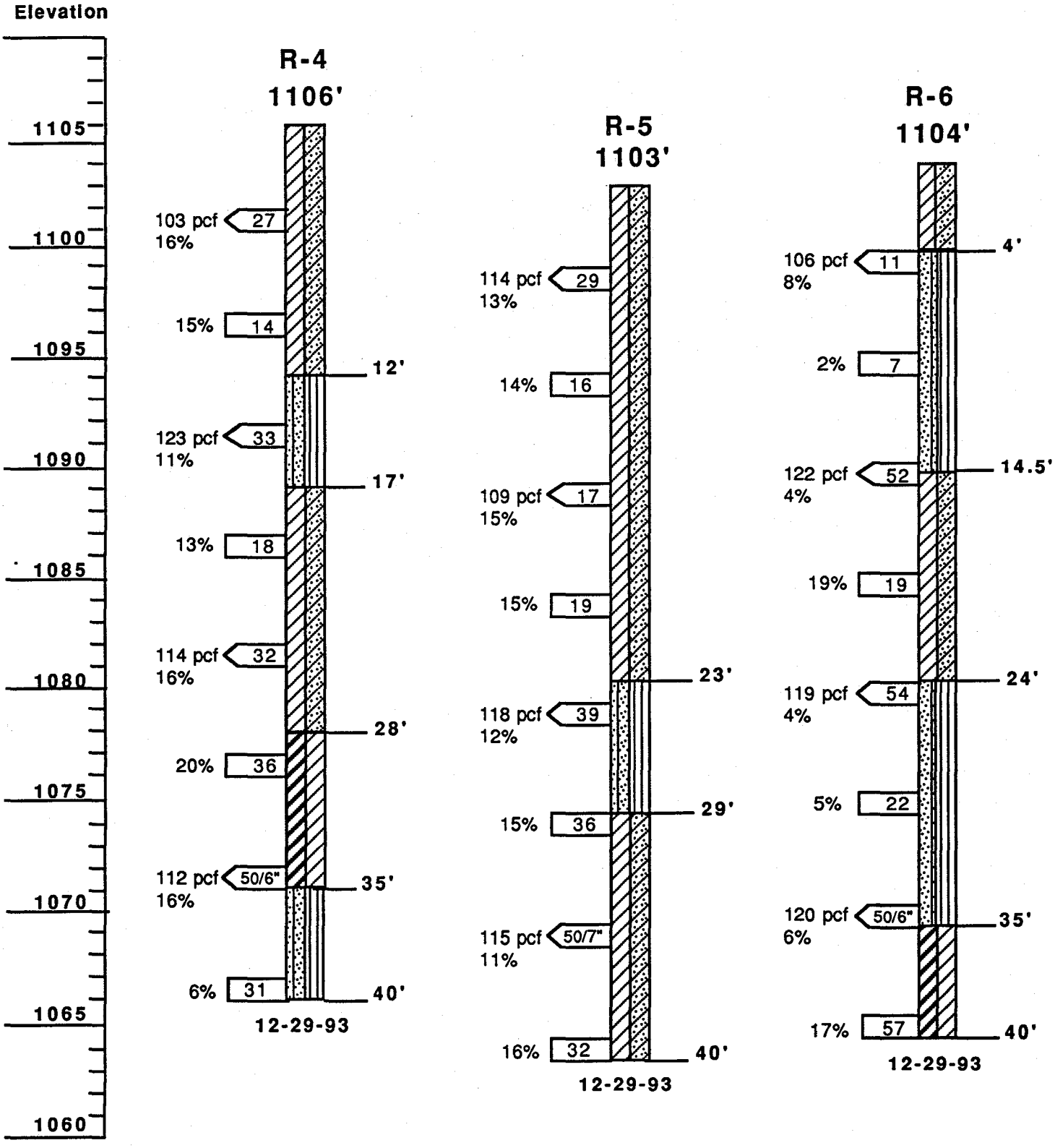

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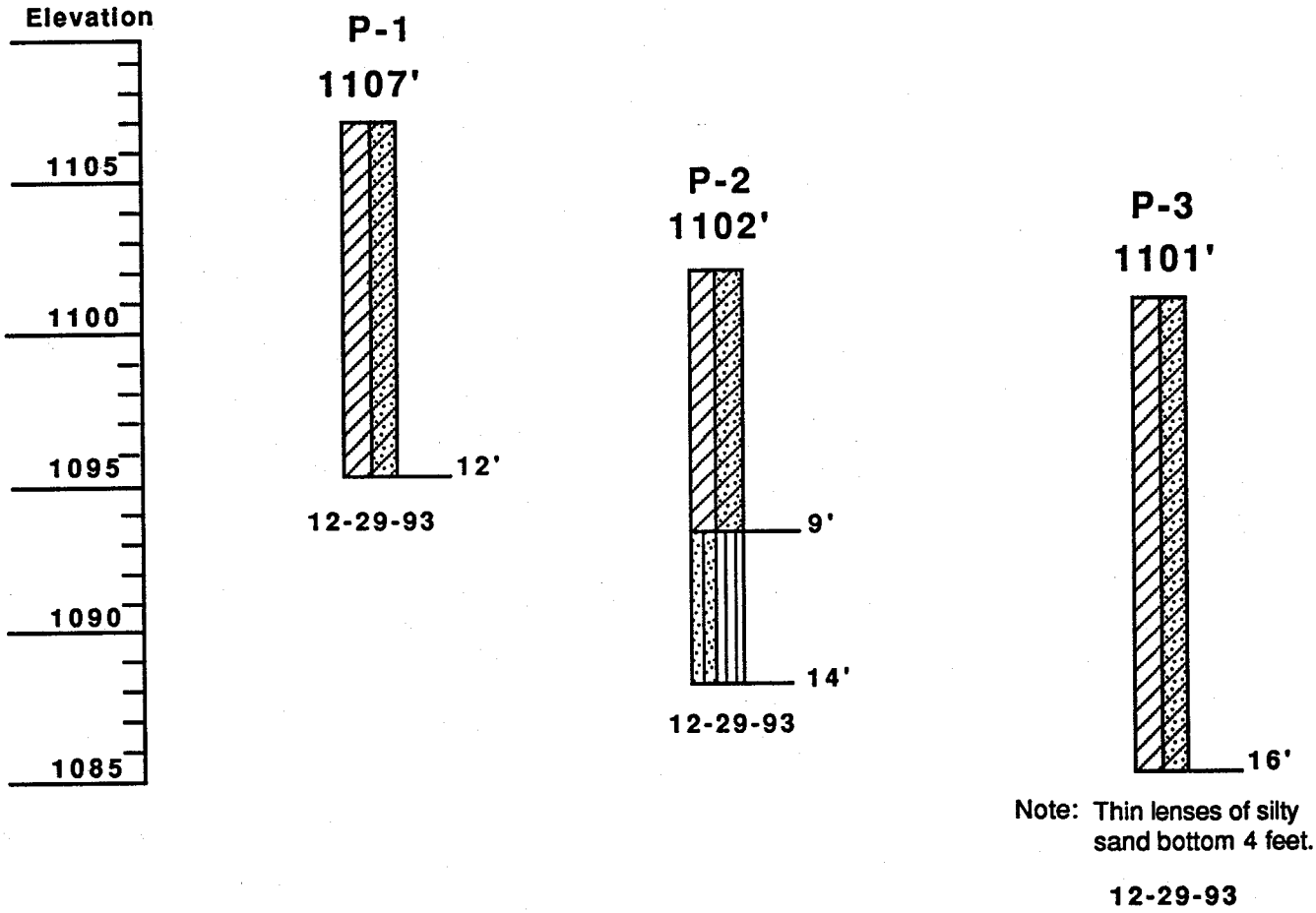


No free groundwater was encountered in any of the borings during drilling.

All borings drilled with 7" diameter hollow stem auger unless otherwise noted.

NOTE: The data presented on the boring logs represents subsurface conditions only at the specific locations and at the time designated. This data may not represent conditions at other locations and/or times. Contacts between soil strata are approximate and changes between soil types may be gradual rather than abrupt. This boring data was compiled primarily for design purposes and should not be construed as part of the plans governing construction or defining construction techniques. Bidders are fully responsible for interpretations or conclusions they draw from the boring log.

Project No. 93-1033
Thomas-Hartig & Associates

SOIL LEGEND

SAMPLE DESIGNATION	DESCRIPTION	
AS	Auger Sample-	A grab sample taken directly from auger flights
BS	Large Bulk Sample-	A grab sample taken directly from auger flights
S	Spoon Sample-	Standard Penetration Test (ASTM D-1586) Driving a 2.0 inch outside diameter split spoon sampler into undisturbed soil for three successive 6-inch increments by means of a 140 lb. weight free falling through a distance of 30 inches. The cumulative number of blows for the final 12 inches of penetration is the Standard Penetration Resistance (N).
RS	Ring Sample-	Driving a 3.0 inch outside diameter spoon equipped with a series of 2.42 inch inside diameter, 1 inch long brass rings, into undisturbed soil for one 12 inch increment by the same means of the Spoon Sample. The blows required for the 12 inches of penetration are recorded.
LS	Liner Sample-	Standard Penetration Test driving a 2.0 inch outside diameter split spoon equipped with two 3 inch long, 1⅜ inch inside diameter brass liners, separated by a 1 inch long spacer, into undisturbed soil by the same means of the spoon sample.
ST	Shelby Tube-	A 3.0 inch outside diameter thin-walled tube continuously pushed into undisturbed soil by a rapid motion, without impact or twisting (ASTM D-1587).
–	Continuous Penetration Resistance-	Driving a 2.0 inch outside diameter "Bullnose Penetrometer" continuously into undisturbed soil by the same means of the spoon sample. The blows for each successive 12 inch increment are recorded.

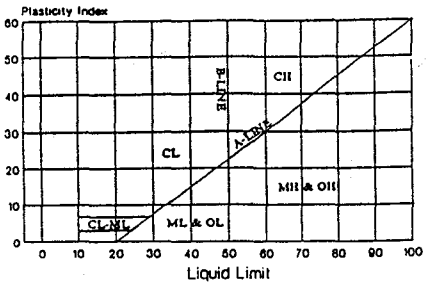
CONSISTENCY			RELATIVE DENSITY	
Clays & Silts	Blows/Foot *	Strength †	Sands and Gravels	Blows/Foot *
Very Soft	0-2	0-¼	Very Loose	0-4
Soft	2-4	¼-½	Loose	5-10
Firm	5-8	½-1	Medium Dense	11-30
Stiff	9-15	1-2	Dense	31-50
Very Stiff	16-30	2-4	Very Dense	> 50
Hard	> 30	> 4		

* Number of blows of a 140 lb hammer free falling 30 inches to drive a 2 inch O.D. split spoon sampler (ASTM D-1588)
† Unconfined compressive strength in tons/sq ft. Read from a pocket penetrometer

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
Coarse Grained Soils	Gravel and Gravelly Soils	Clean Gravels		GW	Well Graded Gravels
				GP	Poorly Graded Gravels
More than 50% of material is larger than #200 sieve size	50% Coarse Fraction is > #4 Sieve Size	Gravels w/Fines		GM	Silty Gravels
				GC	Clayey Gravels
	Sand and Sandy Soils	Clean Sands		SW	Well Graded Sands
				SP	Poorly Graded Sands
	50% Coarse Fraction is < #4 Sieve Size	Sands w/Fines		SM	Silty Sand
				SC	Clayey Sand
Fine Grained Soils	Silts and Clays			ML	Inorganic Silts, Low Plasticity
	Liquid limit is less than 50%			CL	Inorganic Clays, Low Plasticity
				OL	Organic Silts, High Plasticity
More than 50% of material is smaller than #200 sieve size	Silts and Clays			MH	Inorganic Silts, High Plasticity
	Liquid limit is greater than 50%			CH	Inorganic Clays, High Plasticity
				OH	Organic Clays, High Plasticity
Highly Organic Soils				PT	Peat and Humus, Highly Organic

MATERIAL SIZE	PARTICLE SIZE			
	Lower Limit		Upper Limit	
	mm	Sieve Size †	mm	Sieve Size †
Sands				
Fine	.075	#200	0.42	#40
Medium	0.42	#40	2.00	#10
Coarse	2.00	#10	4.76	#4
Gravels				
Fine	4.76	#4	191	¾" °
Coarse	191	¾" °	762	3" °
Cobbles	762	3" °	304.8	12" °
Boulders	304.8	12" °	914.4	36" °

†U.S. Standard °Clear Square Openings



Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 165+20, CL

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	6" Concrete					
0.5	Very Dense Light Brown GRAVELLY SAND (SW-Dry)	RS-1	2.0	--	--	
2.5	Hard Brown SANDY CLAY (CL-Dry) With a Trace of Gravel	S-2	5.0	--	--	
5						
7.5	Hard Brown SANDY SILT (ML-Dry)					
10	End of Boring	S-3	10.0	--	--	57/12"
15						

Boring Date: 2-19-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B- 1

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 181+90, CL

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)
0	6" Concrete				
0.5	Medium dense Brown SAND (SW-Dry)	S-1	2.5	--	--
3.0	Dense Brown SILTY SAND (SM-Dry)	RS-2	4.5	5.4	133.2
5					
6.5	Very Stiff to Hard Brown CLAY (CL-Dry) With a Little Calcareous Cementation; Heavily Cemented at 9.5 ′				
10	End of Boring	S-3	10.0	--	--
15					

Penetration Resistance Blows per Foot: 0 25 50

Boring Date: 2-19-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B- 2

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 162+25, CL

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	6" Concrete (0.5)					
	Very Stiff Light Brown SANDY CLAY (CH-Moist) (3.0)	S-1	2.5	--	--	
	Medium Dense Brown CLAYEY SAND (SC-Moist) (5.0)	S-2	5.0	--	--	
5	Hard Slightly Mottled Brown and White SANDY CLAY (CH-Dry) With a Little Calcareous Cementation (9.5)	S-3	9.5	--	--	71/12"
	End of Boring					
10						
15						

Boring Date: 3-24-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B- 3

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 173+00, CL

Depth (feet)	Visual Classification		Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	6" Concrete	0.5					
	Medium Dense Brown SILTY SAND (SM-Moist)	3.0	S-1	2.5	--	--	
	Medium Dense Brown CLAYEY SAND (SC-Moist)	4.5					
5	Very Stiff Brown SANDY CLAY (CL-Moist)	6.5	S-2	5.0	--	--	
	Hard Brown SILTY CLAY (CL-Moist) With a Trace of Sand and Some Mild Calcareous Cementation	10.0	S-3	10.0	--	--	
10	End of Boring						
15							

Boring Date: 3-24-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B- 4

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 186+70, CL

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	6" Concrete					
0.5	Medium Dense Brown CLAYEY SAND (SC-Moist)					
2.5		S-1	2.5	--	--	
	Medium Dense Brown CLAYEY COARSE SAND (SC-Dry)					
5.0		S-2	5.0	--	--	
5	Hard Brown SILTY CLAY (CL-Dry) With Small Veins of Calcareous Cementation					
10.0		S-3	10.0	--	--	50/12"
10	End of Boring					
15						

Boring Date: 3-24-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B- 5

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 118+80

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	Stiff Brown SANDY SILT (ML-Dry)	RS-1	2.5	6.2	117.8	
3.0	Interbedded Loose and Stiff SILTY SAND & SANDY CLAY (SM/CL-Dry)	S-2	6.0	--	--	
5						
7.5	Hard Mottled Brown/Light Brown SANDY CLAY (CL-Dry to Moist) With Some Weak Calcareous Cementation					
10						
11.0	End of Boring	S-3	11.0	--	--	
15						

Boring Date: 4-2-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B- 6

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 110+00

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	Stiff Brown SANDY SILT (CL-Dry)	RS-1	2.5	11.3	109.9	
3.5	Medium Dense Light Brown CLAYEY SAND (SC-Dry) With a Trace of Cementation	S-2	6.0	--	--	
5						
7.5	Hard Light Brown CLAY (CL-Dry) With a Little Calcareous Cementation					
10						
11.0	End of Boring	S-3	11.0	--	--	91/12"
15						

Boring Date: 4-2-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B- 7

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Rig Type: **CME-55**
Boring Type: **Hollow Stem Auger**
Surface Elevation: **N/A**
Sta. 100+00

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	Hard Brown Fine SANDY CLAY (CL-Dry) With a Trace of Calcareous Cementation	RS-1	2.4	--	--	50/11"
3.0	Very Stiff Light Brown CLAY/SILT (CL/ML) With a Trace of Fine Sand and Calcareous Cementation	BS-2	4.0	--	--	
5		S-3	6.0	--	--	
7.5	Hard Light Brown CLAY (CL-Dry) With Some Calcareous Cementation					
10		S-4	11.0	--	--	81/11"
10.4	End of Boring					
15						

Boring Date: **4-2-93**
Field Engineer/Technician: **C. Spencer**
Driller: **S. Belsky**
Contractor: **Heber Mining**

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: **B- 8**

Dysart Drain

Dysart Road N/O Glendale Avenue

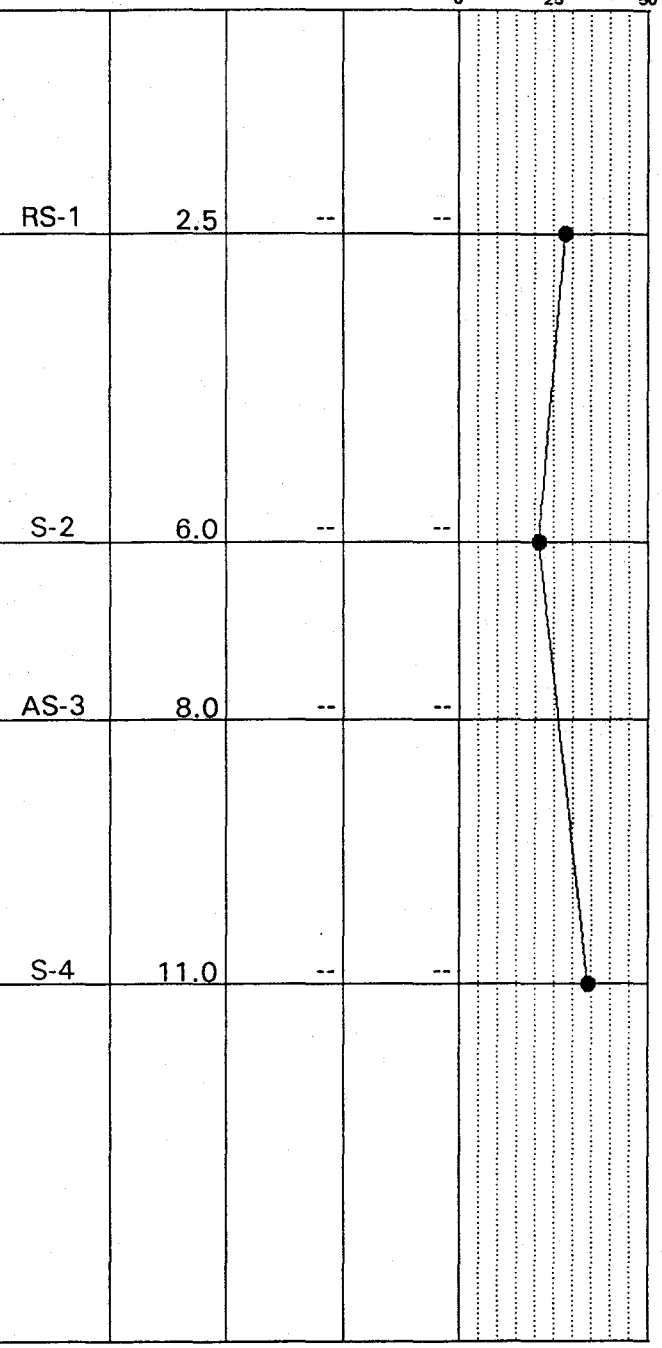
Maricopa County, Arizona

Project No.: **92113SA1**

Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 90+00

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)
0	Very Stiff Brown Fine SANDY CLAY (CL-Dry to Moist) With a trace of Calcareous Cementation	RS-1	2.5	--	--
5		S-2	6.0	--	--
6.0	Very Stiff White CLAY (CL-Moist) With a Trace of Sand	AS-3	8.0	--	--
9.5	Hard Brown SILTY CLAY (CL-Dry)	S-4	11.0	--	--
10					
11.0	End of Boring				
15					

Penetration Resistance Blows per Foot (0, 25, 50)



Boring Date: 4-2-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B- 9

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Rig Type: CME-55
Boring Type: Hollow Stem Auger
Surface Elevation: N/A
Sta. 80+00

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	Firm Brown Fine SANDY CLAY (CL-Moist)	RS-1	2.5			
3.0	Hard Light Brown SANDY CLAY CL-Dry) With Some Calcareous Cementation	S-2	6.0	--	--	
5						
7.5	Hard Mottled Brown, Green, Gray, Light Brown and Reddish Brown CLAY (CH-Moist)					
10						
11.0	End of Boring	S-3	11.0	--	--	
15						

Boring Date: 4-2-93
Field Engineer/Technician: C. Spencer
Driller: S. Belsky
Contractor: Heber Mining

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Boring Number: B-10

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

Equipment: John Deere 510B
Extend-a-hoe

Surface Elevation: N/A
Sta 180

Depth (feet)	Visual Classification	Sample Number	Depth of Sample	Natural Water Content (%)	In-place Dry Density (P.C.F.)	Penetration Resistance Blows per Foot (0 – 25 – 50)
0	8" Concrete					
0.7	Stiff Brown SANDY CLAY (CH-Dry to Moist)	AS-1	2.0	--	--	
3.0	Medium Dense Brown CLAYEY FINE SAND (SC-Dry)	AS-2	5.0	--	--	
5.0	Very Stiff Brown SILTY CLAY/CLAYEY SILT (CL/ML-Dry)					
6.0	End of Test Pit					
10						

Excavation Date: 3-31-93
Field Engineer/Technician: C. Spencer
Operator: MCFC
Contractor: MCFC

Water Level

Depth	Hour	Date

SPEEDIE AND ASSOCIATES

Log of Test Pit Number: TP- 1

Dysart Drain

Dysart Road N/O Glendale Avenue

Maricopa County, Arizona

Project No.: 92113SA1

REPORT ON PERCOLATION TESTS

Date: 12-29-93

Description:

Source: Noted Below

Material: Detention Basin Subsoils

Performed by: TH/Thompson

TESTED: Field percolation tests in uncased 12-inch diameter test borings at indicated depths.

RESULTS:

Location	Depth of Test Boring (feet)	Percolation Rate (min./in.)
P-1	12	1.8
P-2	14	1.8
P-3	16	2.0

Project No. 93-1033

Thomas-Hartig & Associates, Inc.

APPENDIX B

LAB RESULTS

REPORT ON SIEVE ANALYSIS AND PLASTICITY INDEX

SAMPLE: Date: 1-28-94

Source: Noted Below

Type: (S) = SPT Sample (R) = Driven Ring Sample (B) = Bulk Sample

Material: Subsurface Soil

Sampled By: TH/Thompson and Walsh

TESTED: Sieve Analysis and Plasticity Index

RESULTS

			Sieve Size -					Accumulative % Passing						*
Sample	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.
1; 0 - 5' (B)	26	8	75	86	92	95	97	98	99	100				CL
3; 11 - 12' (S)	31	9	93	97	98	99	99	100						CL
4; 9 - 10' (S)	32	11	51	60	67	75	86	96	99	100				CL
5; 0 - 2' (B)	33	15	64	69	73	79	87	93	96	100				CL
10; 4 - 5' (S)	29	12	66	76	84	91	96	98	100					CL
11; 8.5 - 9.5' (R)		NP	23	34	54	78	90	96	100					SM
15; 9 - 10' (S)	57	34	67	71	75	80	85	92	96	100				CH
19; 15 - 20' (B)	89	65	89	92	94	95	97	99	99	100				CH
21; 25 - 35' (B)	41	20	50	58	68	71	73	74	78	94	100			SC-CL
23; 10 - 15' (B)	33	12	41	52	64	75	82	86	91	100				SC
24; 10 - 17' (B)	26	4	11	15	20	26	31	35	39	77	88	100		GW-GM
25; 0 - 5' (B)		NP	7	11	29	50	61	68	72	83	84	100		SP-SM
26; 24 - 25' (S)		NP	4	10	14	22	37	48	59	96	100			SW
R-1; 24 - 25' (S)	33	13	64	76	85	92	97	99	100					CL
R-3; 4 - 5' (S)	28	13	65	76	84	93	98	99	100					CL
R-3; 34 - 35' (S)	31	11	77	86	91	95	97	98	99	100				CL
R-4; 9 - 10' (S)	31	12	80	89	93	96	98	99	100					CL

NP = Non-Plastic

*Unified Soil Classification

Project No. 93-1033

Thomas-Hartig & Associates, Inc.

REPORT ON SIEVE ANALYSIS AND PLASTICITY INDEX

SAMPLE: Date: 1-28-94

Source: Noted Below

Type: (S) = SPT Sample (R) = Driven Ring Sample (B) = Bulk Sample

Material: Subsurface Soil

Sampled By: TH/Thompson and Walsh

TESTED: Sieve Analysis and Plasticity Index

RESULTS

			Sieve Size - Accumulative % Passing											*
Sample	LL	PI	200	100	50	30	16	8	4	3/4"	1"	2"	3"	Class.
B-3; 14 - 15' (R)		NP	7	10	16	35	63	84	92	100				SW-SM
B-10; 20 - 2 (R)	63	40	64	71	77	81	85	90	93	100				CH
B-10; 35 - 36' (S)	41	20	61	68	75	81	88	94	98	100				CL
B-10; 50 - 51' (R)		NP	23	32	56	90	97	99	99	100				SM
B-10; 70 - 71' (R)	48	25	82	88	91	91	92	93	93	94	100			CL
B-16; 10 - 11' (R)	38	13	64	87	98	99	100							CL
B-16; 55 - 56' (S)	30	12	22	28	39	52	61	70	75	100				SC
B-16; 65 - 66' (S)		NP	11	14	18	23	29	37	48	100				GW-GM
B-18; 35 - 36' (R)	33	7	47	62	80	90	93	95	96	100				SC
B-18; 70 - 71' (S)	36	20	25	30	38	45	83	63	73	94	100			SC
E; 0 - 5' (B)	27	11	41						75	100				SC
F; 0 - 5' (B)	41	19	41						84	100				SC

NP = Non-Plastic

*Unified Soil Classification

Project No. 93-1033

Thomas-Hartig & Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

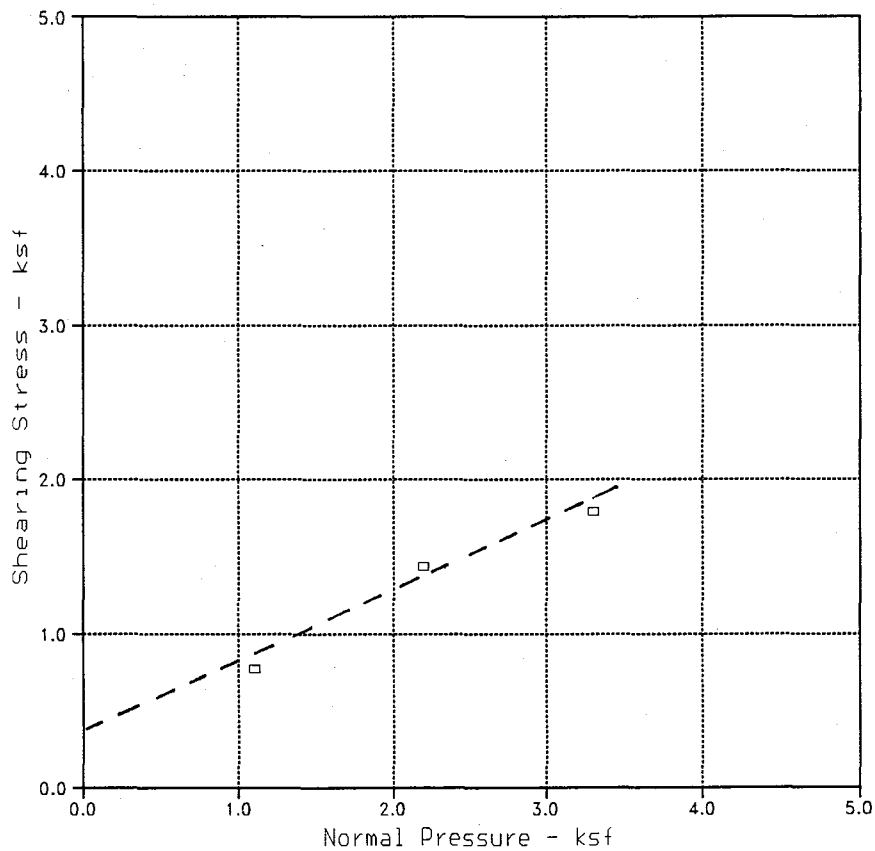
SAMPLE: **Date:** 17-Jan-94

Source: 1 @ 4'-5'
Type: Driven Ring; 97 pcf Dry Density; 9% Field Moisture
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 25 deg. Cohesion (c) = 0.35 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

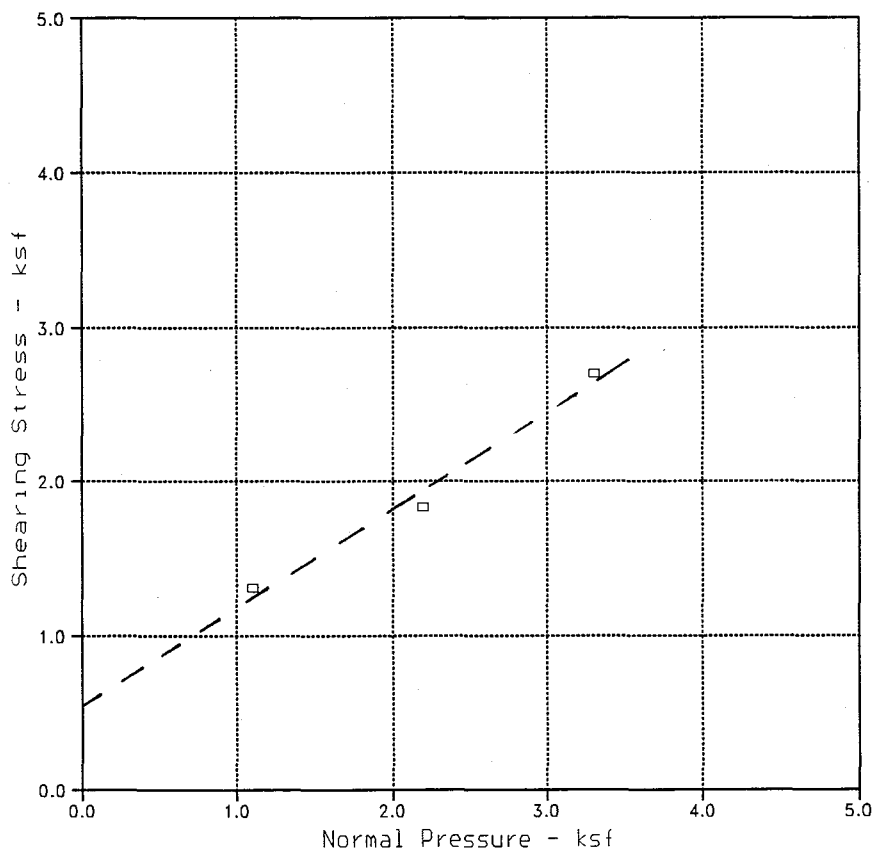
SAMPLE: **Date:** 17-Jan-94

Source: 4 @ 4'-5'
Type: Driven Ring; 120 pcf Dry Density; 6% Field Moisture
Material: Silty Sand and Gravel (SM-GM)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 33 deg. Cohesion (c) = 0.55 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

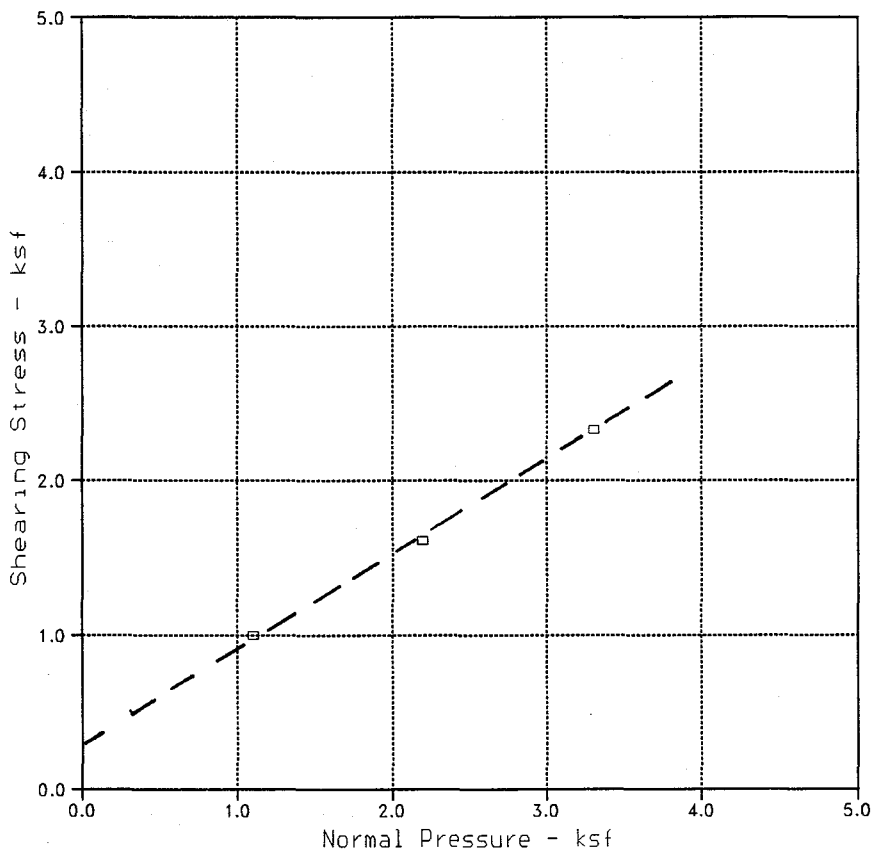
SAMPLE: **Date:** 04-Jan-94

Source: 11 @ 9.5'-10.5'
Type: Driven Ring; 120 pcf Dry Density; 6% Field Moisture
Material: Silty Sand (SM)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 32 deg. Cohesion (c) = 0.30 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

SAMPLE: **Date:** 20-Dec-93

Source: 17 @ 4'-5'

Type: Driven Ring; 111 pcf Dry Density; 16% Field Moisture

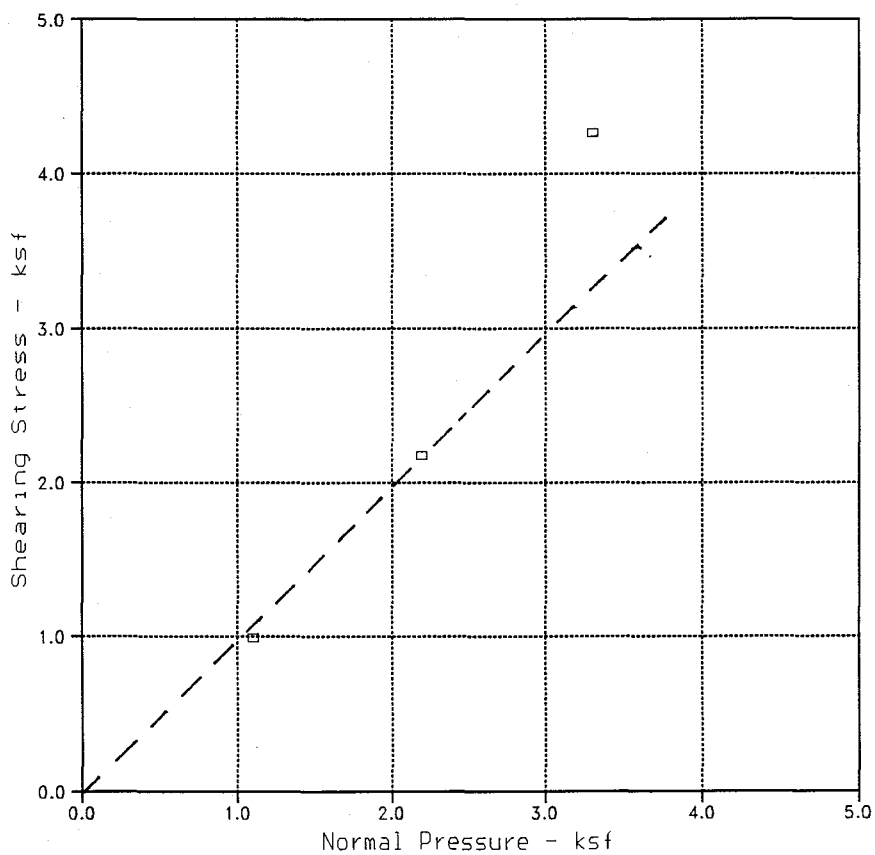
Material: Sandy Clay (CL)

Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 45 deg. Cohesion (c) = 0.0 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

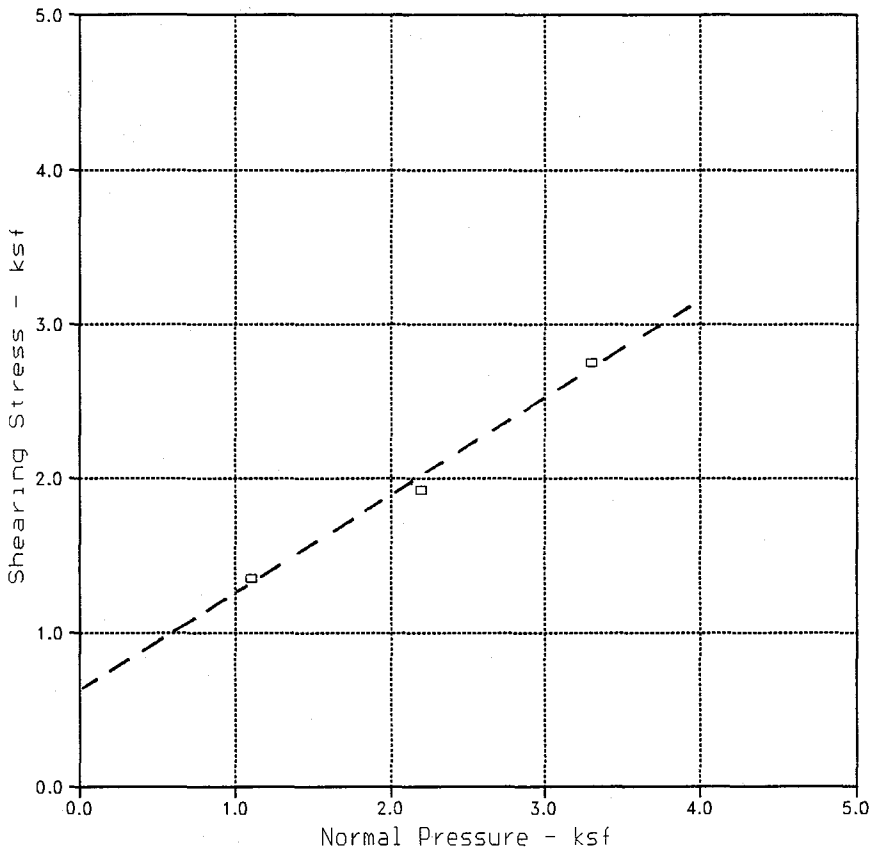
SAMPLE: **Date:** 20-Dec-93

Source: 18 @ 19'-20'
Type: Driven Ring; 101 pcf Dry Density; 23% Field Moisture
Material: Sandy Silt (MH)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 32 deg. Cohesion (c) = 0.65 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

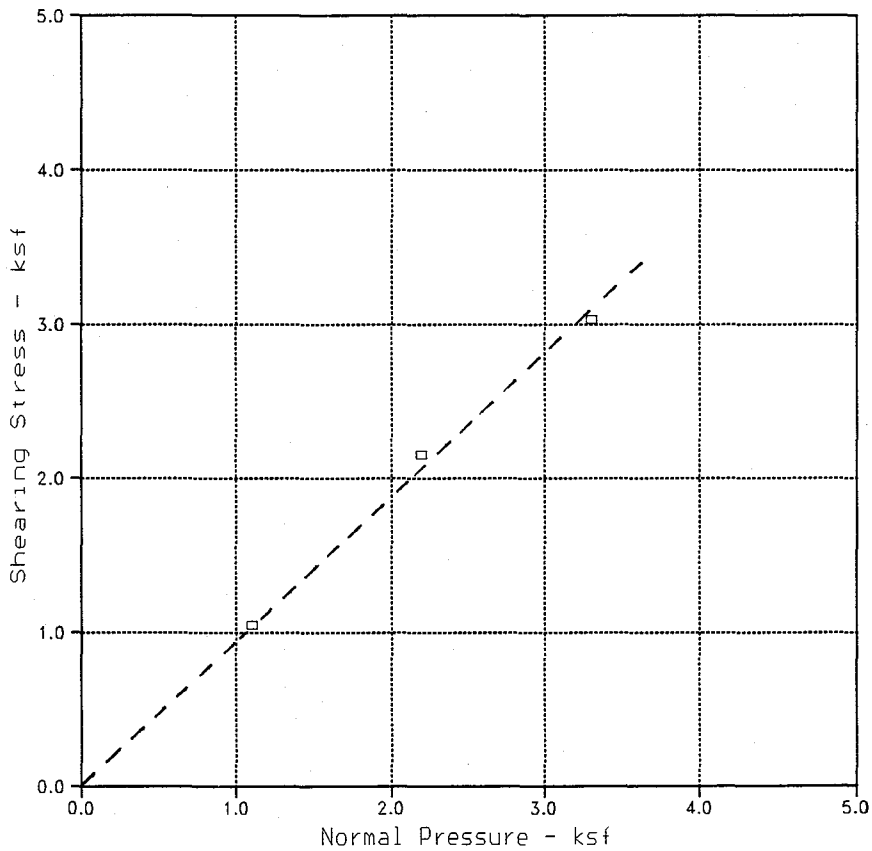
SAMPLE: **Date:** 20-Dec-93

Source: 23 @ 14'-15'
Type: Driven Ring; 121 pcf Dry Density; 4% Field Moisture
Material: Silty Sand (SM)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 43 deg. Cohesion (c) = 0.00 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

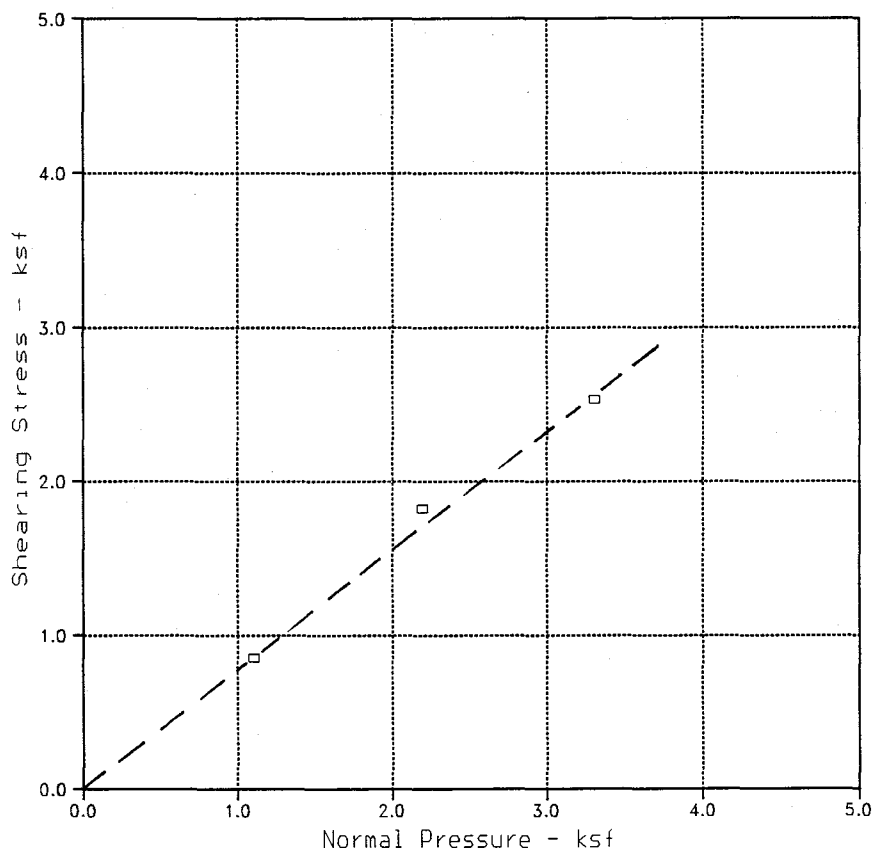
SAMPLE: **Date:** 17-Jan-94

Source: 25 @ 9'-10'
Type: Driven Ring; 124 pcf Dry Density; 4% Field Moisture
Material: Silty Sand (SP-SM)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 37 deg. Cohesion (c) = 0.0 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

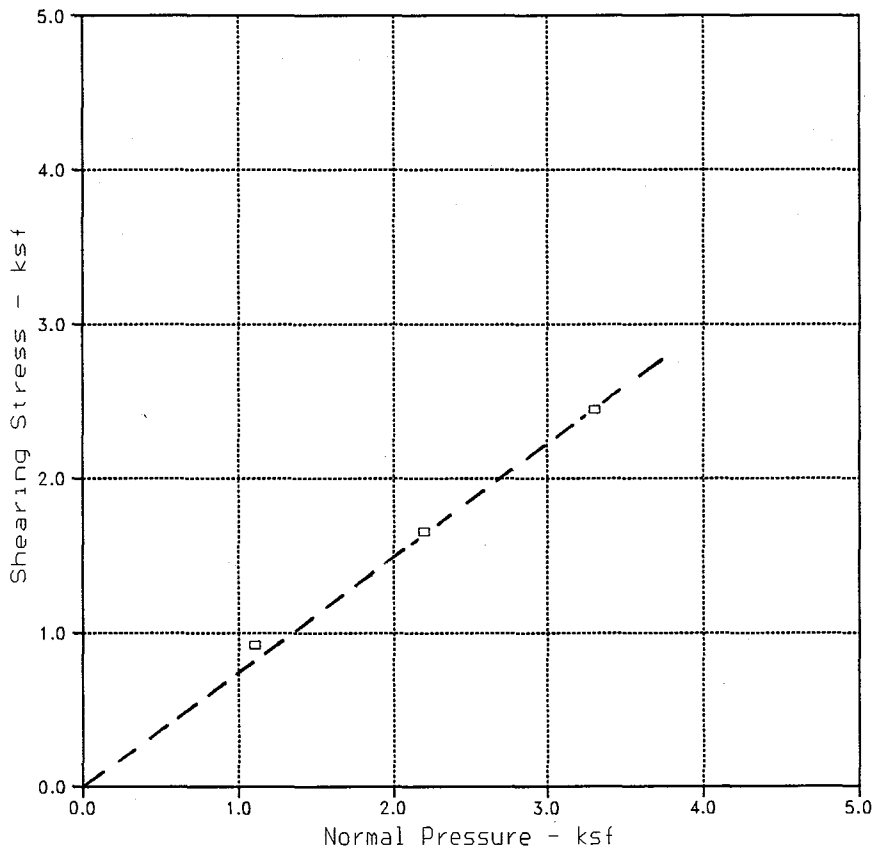
SAMPLE: **Date:** 17-Jan-94

Source: 26 @ 19'-20'
Type: Driven Ring; 112 pcf Dry Density; 5% Field Moisture
Material: Sand and Gravel (SW-GW)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 37 deg. Cohesion (c) = 0.0 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

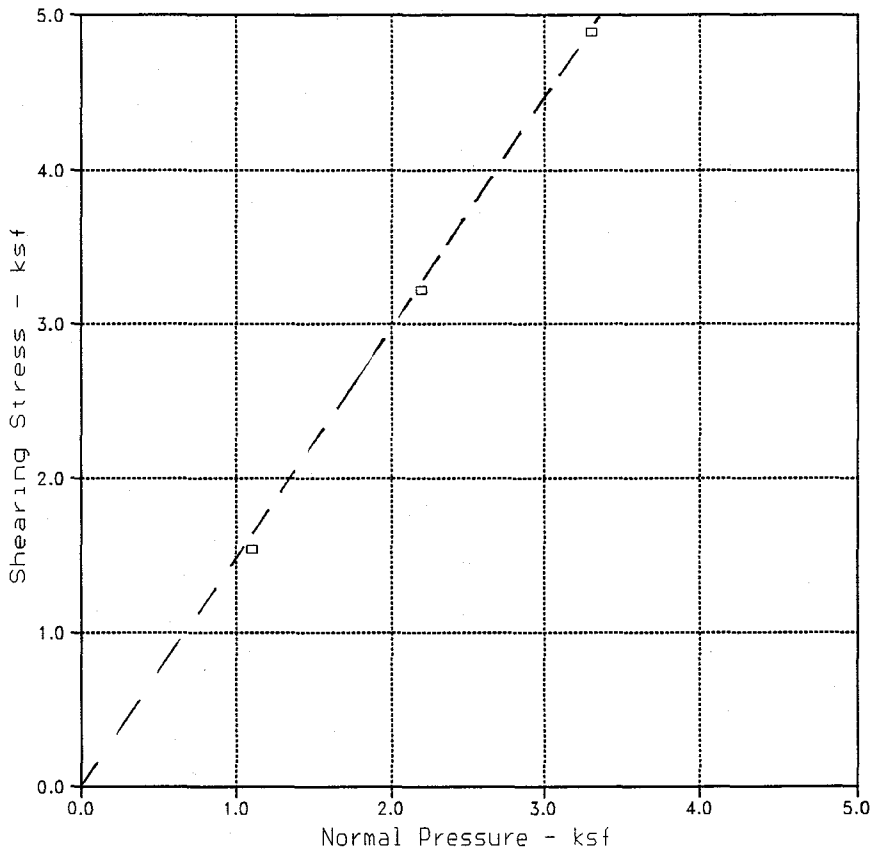
SAMPLE: **Date:** 27-Jan-94

Source: B10 @ 20'-21'
Type: Driven Ring; 107 pcf Dry Density; 18% Field Moisture
Material: Clay (CH)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 55 deg. Cohesion (c) = 0.00 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

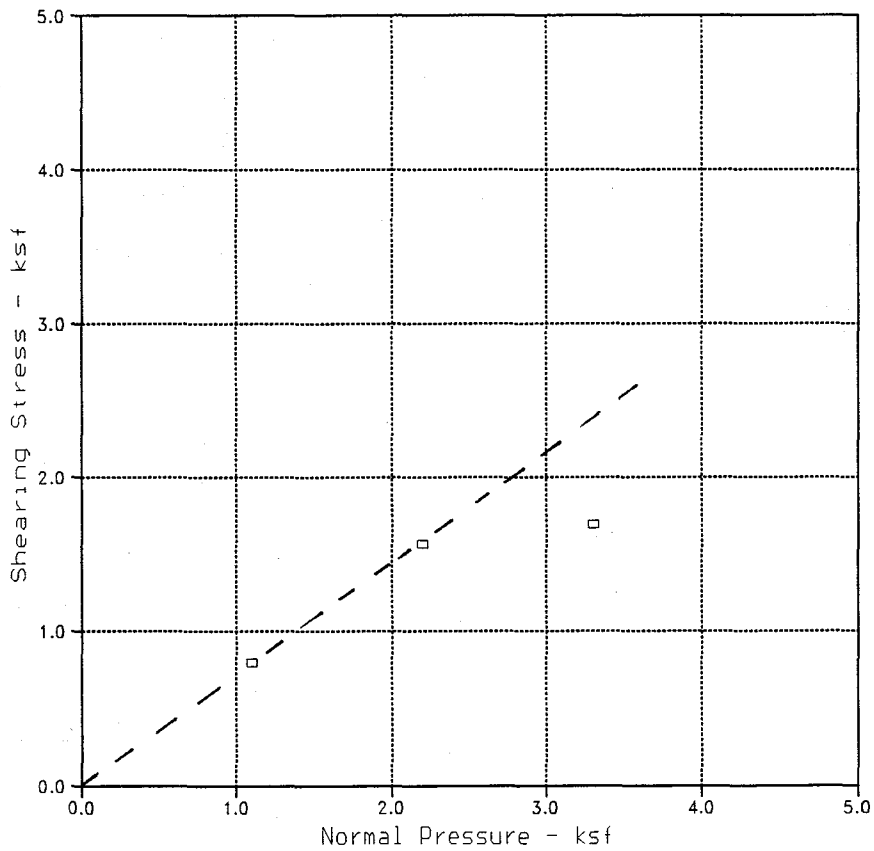
SAMPLE: **Date:** 27-Jan-94

Source: B10 @ 50'-51'
Type: Driven Ring; 93 pcf Dry Density; 7% Field Moisture
Material: Silty Sand (SM)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 35 deg. Cohesion (c) = 0.0 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

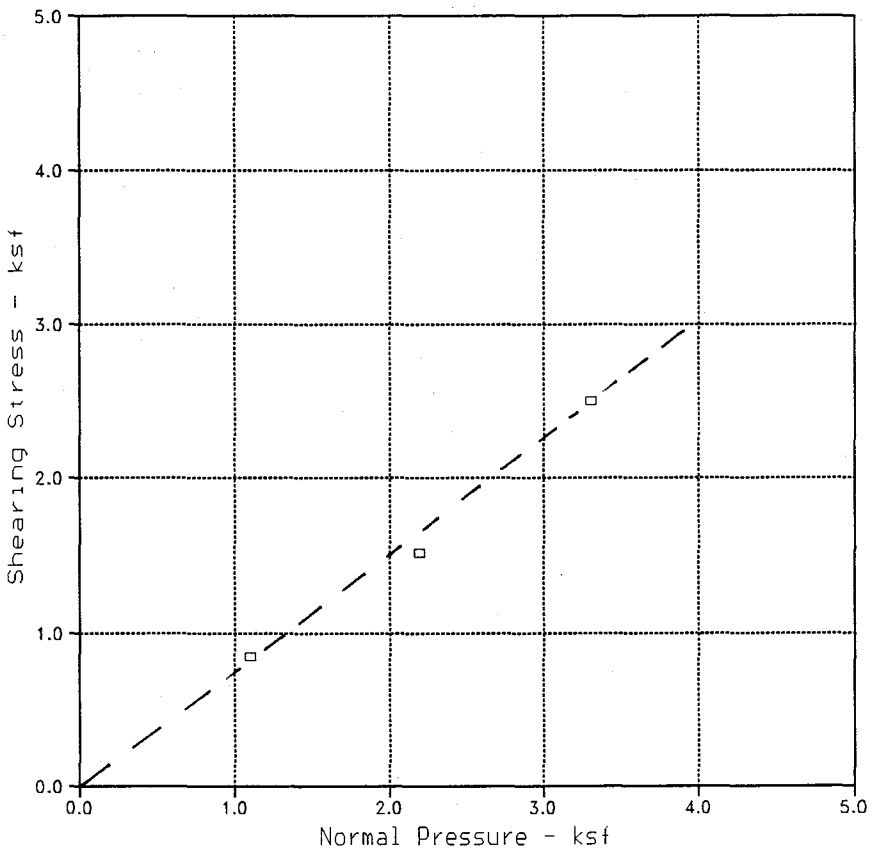
SAMPLE: **Date:** 27-Jan-94

Source: B16 @ 10'-11'
Type: Driven Ring; 104 pcf Dry Density; 8% Field Moisture
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 36 deg. Cohesion (c) = 0.0 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

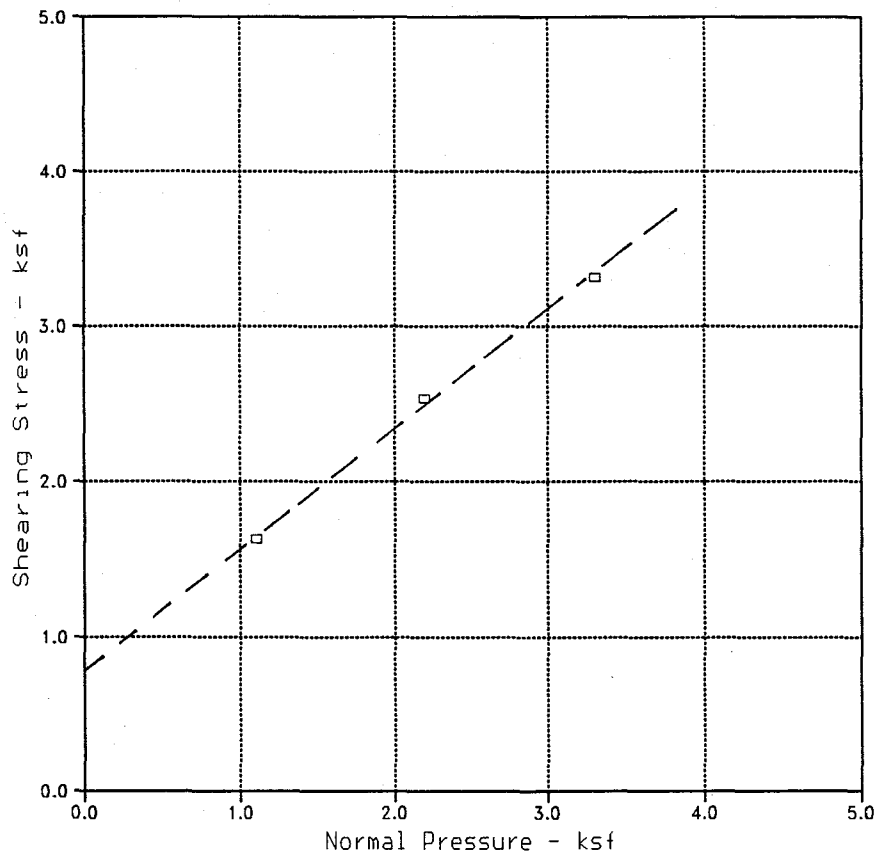
SAMPLE: **Date:** 27-Jan-94

Source: B18 @ 35'-36'
Type: Driven Ring; 112 pcf Dry Density; 9% Field Moisture
Material: Clayey Sand (SC)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 38 deg. Cohesion (c) = 0.85 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

RESULTS OF DIRECT SHEAR TESTS

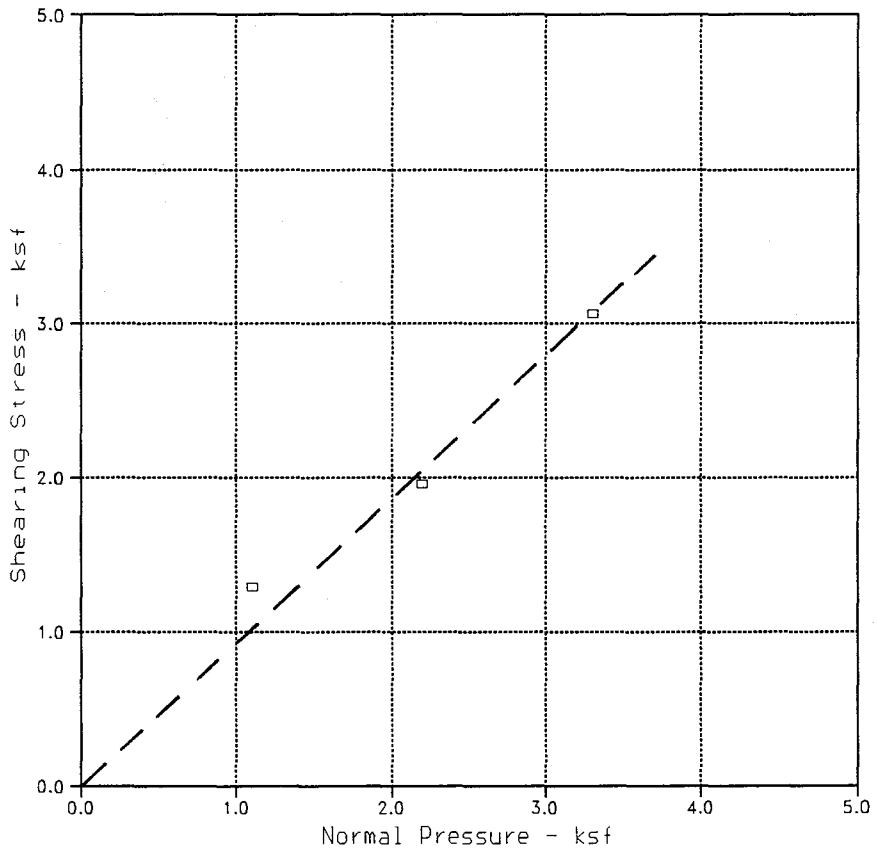
SAMPLE: **Date:** 24-Jan-94

Source: R3 @ 9'-10'
Type: Driven Ring; 108 pcf Dry Density; 16% Field Moisture
Material: Silty Sand (SM)
Sampled By: TH/Thompson

TESTED: ASTM D3080; Samples soaked.

RESULTS:

Friction Angle (phi) = 43 deg. Cohesion (c) = 0.0 ksf



Project No. 93-1033

Thomas-Hartig and Associates, Inc.

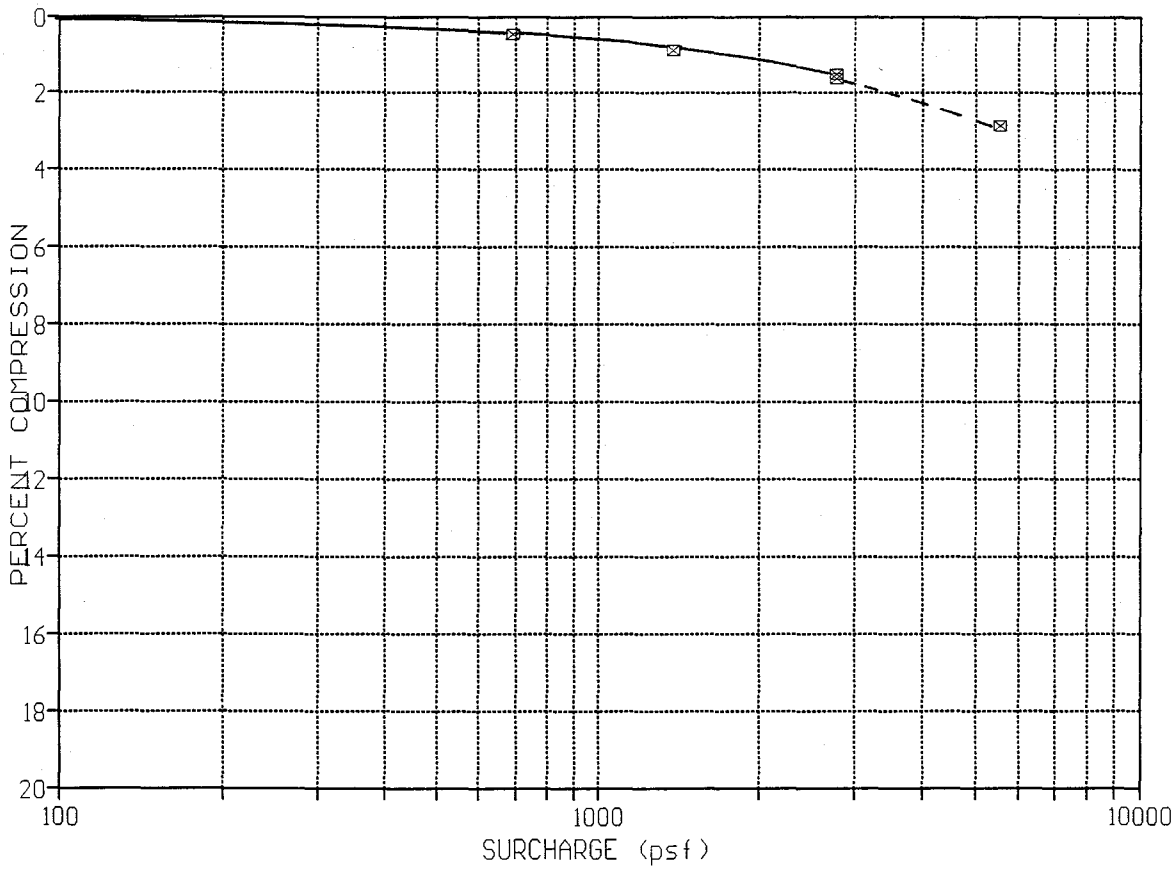
RESULTS OF CONSOLIDATION TESTS

SAMPLE: **Date:** 27-Jan-94

Source: B2 @ 14'-15'
Type: Driven Ring; 107 pcf Dry Density; 16% Field Moisture
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

TESTED: ASTM D2435 : Sample soaked at 2770 psf.

RESULTS:



Project No.: 93-1033

Thomas-Hartig and Associates, Inc.

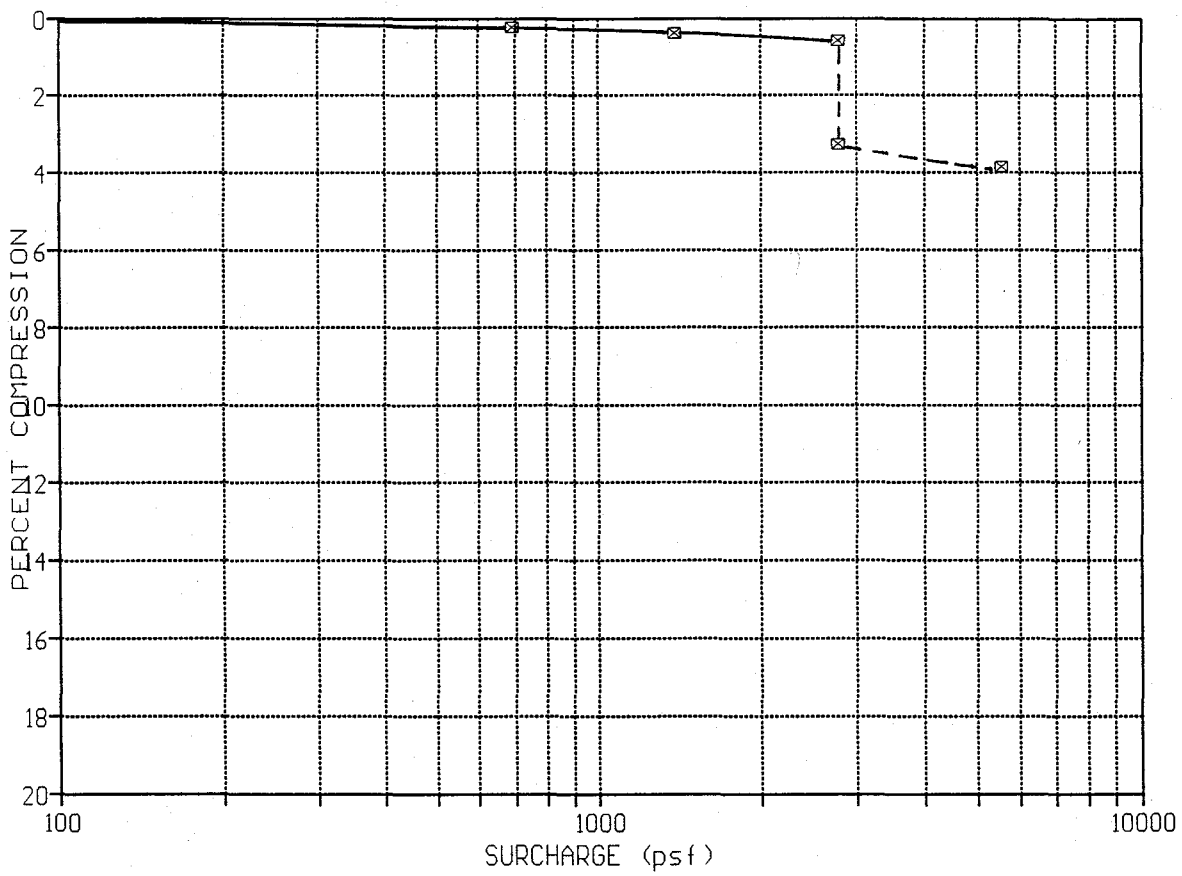
RESULTS OF CONSOLIDATION TESTS

SAMPLE: **Date:** 17-Jan-94

Source: B7 @ 9'-10'
Type: Driven Ring; 122 pcf Dry Density; 5% Field Moisture
Material: Silty Sand (SM)
Sampled By: TH/Thompson

TESTED: ASTM D2435 : Sample soaked at 2770 psf.

RESULTS:



Project No.: 93-1033

Thomas-Hartig and Associates, Inc.

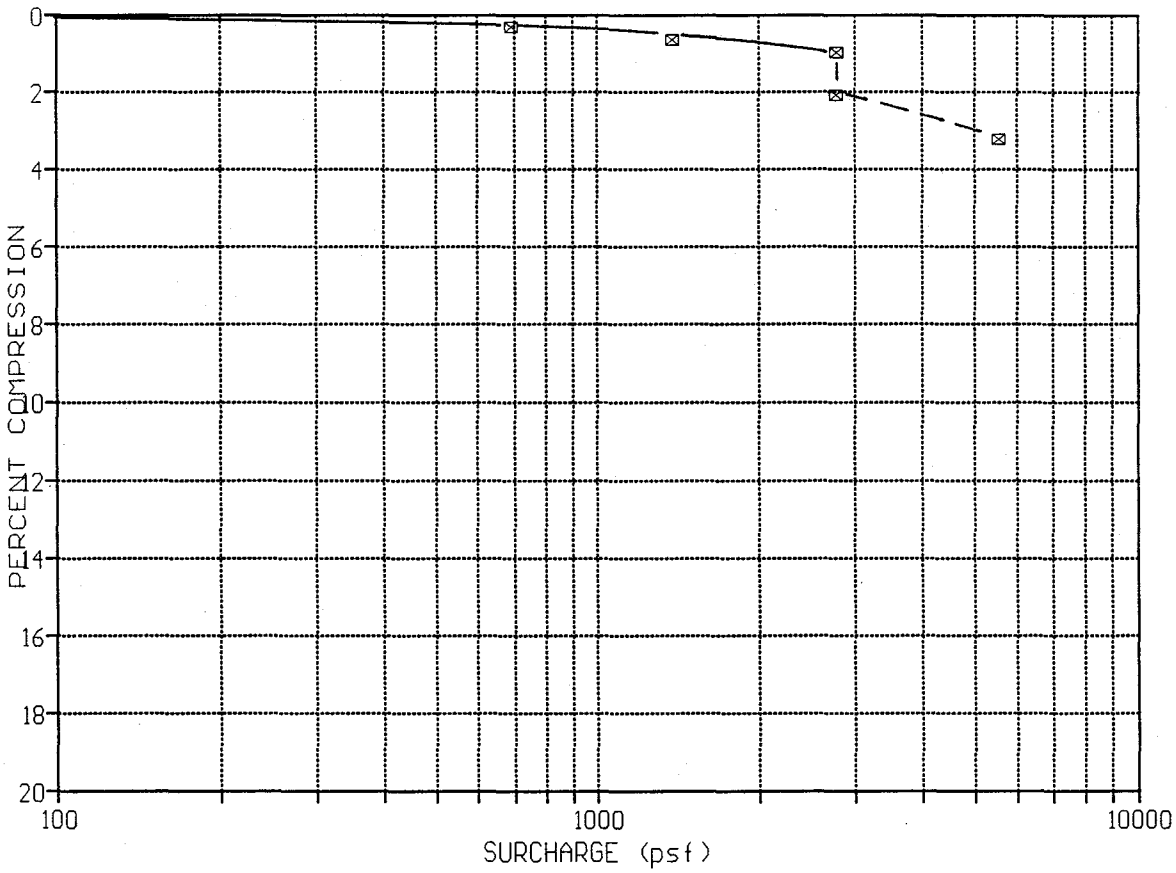
RESULTS OF CONSOLIDATION TESTS

SAMPLE: **Date:** 27-Jan-94

Source: B10 @ 60'-61'
Type: Driven Ring; 121 pcf Dry Density; 10% Field Moisture
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

TESTED: ASTM D2435 : Sample soaked at 2770 psf.

RESULTS:



Project No.: 93-1033

Thomas-Hartig and Associates, Inc.

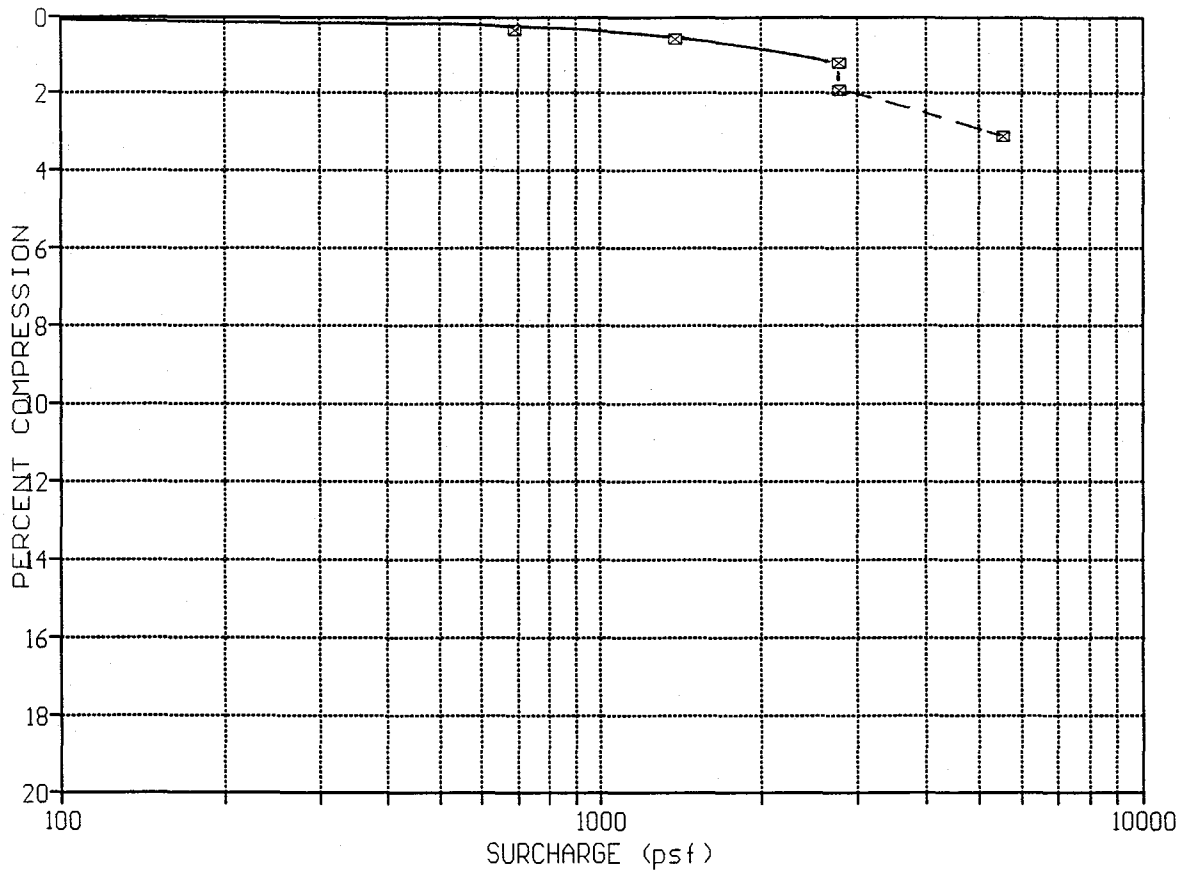
RESULTS OF CONSOLIDATION TESTS

SAMPLE: **Date:** 27-Jan-94

Source: B10 @ 30'-31'
Type: Driven Ring; 94 pcf Dry Density; 17% Field Moisture
Material: Silty Sand (SM)
Sampled By: TH/Thompson

TESTED: ASTM D2435 : Sample soaked at 2770 psf.

RESULTS:



Project No.: 93-1033

Thomas-Hartig and Associates, Inc.

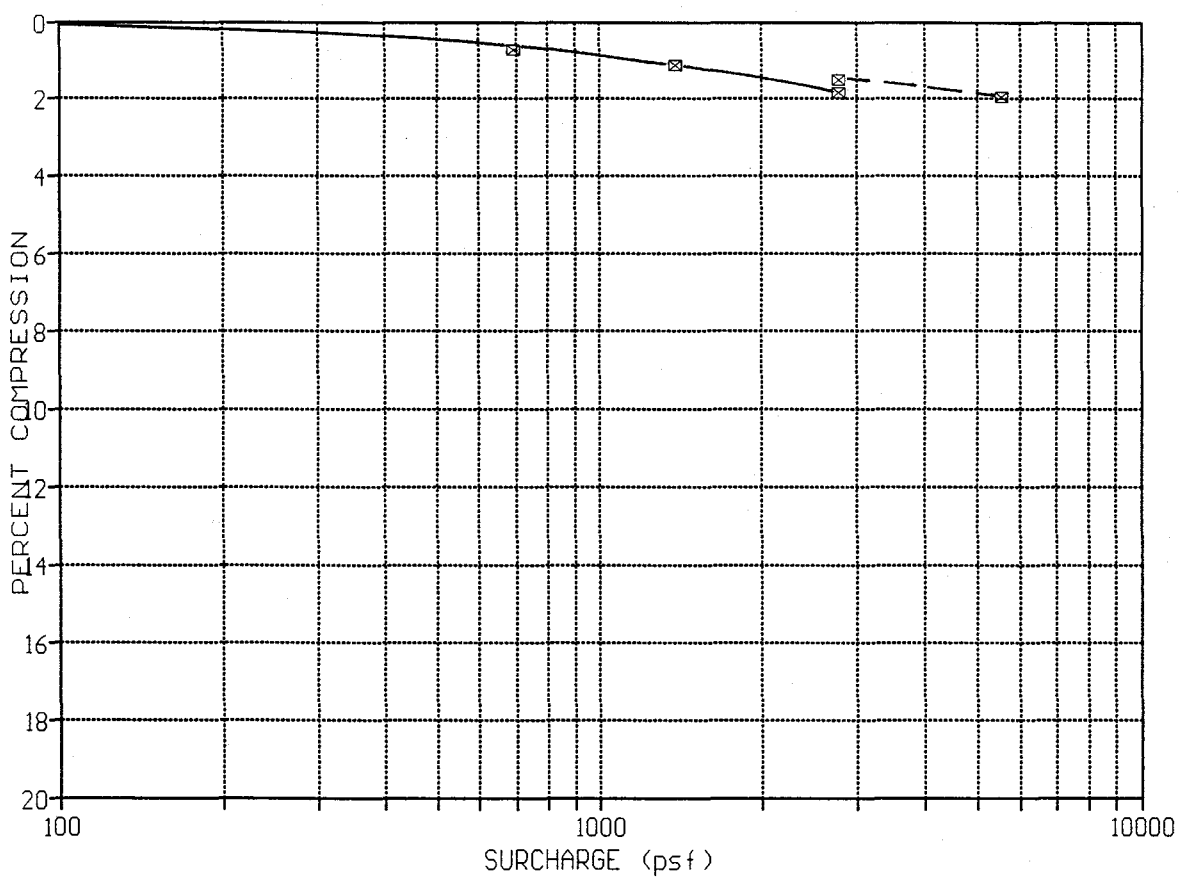
RESULTS OF CONSOLIDATION TESTS

SAMPLE: **Date:** 27-Jan-94

Source: B10 @ 10'-11'
Type: Driven Ring; 97 pcf Dry Density; 25% Field Moisture
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

TESTED: ASTM D2435 : Sample soaked at 2770 psf.

RESULTS:



Project No.: 93-1033

Thomas-Hartig and Associates, Inc.

REPORT ON SOLUBLE SALTS & SULFATES

Date: 1-27-94

Description:

Source: Noted Below
Type: Bulk Samples
Material: Subsurface Soils
Performed by: TH/Thompson

TESTED: Soluble Salts & Sulfates

RESULTS:

Sample	Soluble Salts (%)	Sulfates Percent
B-10; 5 - 15'	0.11	0.00%
B-10; 40 - 50'	0.14	0.060
B-16; 15 - 25'	0.12	0.083
B-18; 20 - 25'	0.17	0.030

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Project No. 93-1033

Thomas-Hartig & Associates, Inc.

REPORT ON pH AND RESISTIVITY TESTS

Date: 1-27-94

Description:

Source: Noted Below

Type: Bulk Samples

Material: Subsurface Soil

Performed by: TH/Thompson

TESTED: pH and Minimum Resistivity (ADOT 236)

TEST RESULTS:

Sample	pH	Minimum Resistivity (ohm-cm)
B-10; 5 - 15'	8.15	620
B-10; 40 - 50'	9.34	540
B-16; 15 - 25'	8.83	610
B-18; 20 - 25'	9.55	610

Project No. 93-1033

Thomas-Hartig & Associates, Inc.

MAXIMUM DENSITY-OPTIMUM MOISTURE

(ASTM D698, METHOD A)

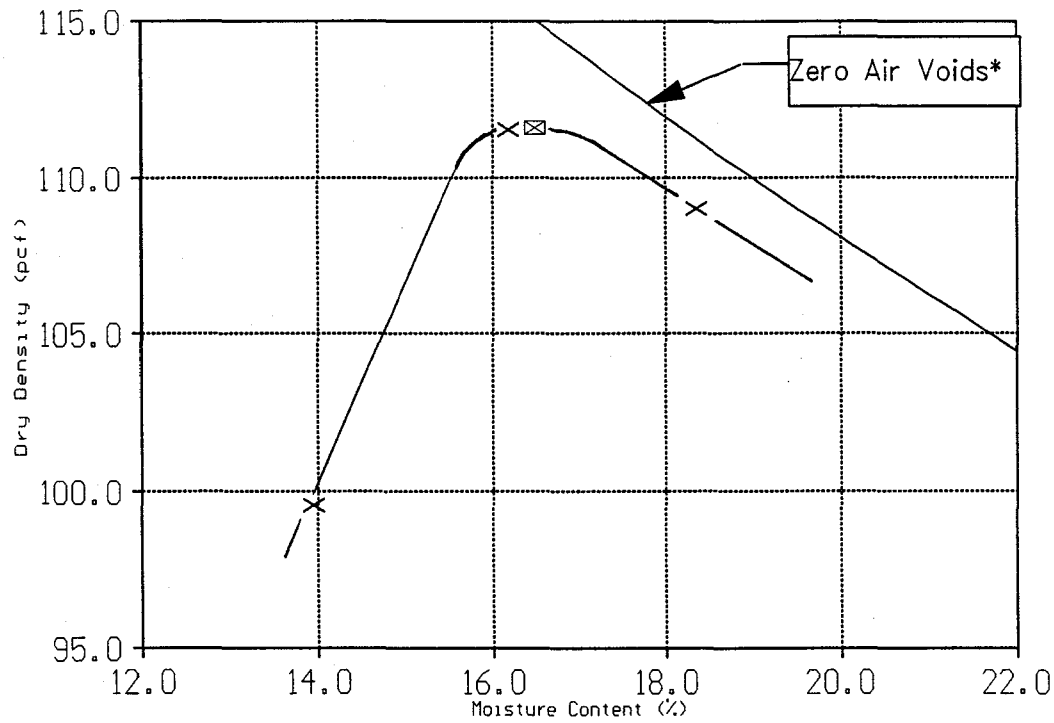
SAMPLE: **Date:** 20-Jan-94

Source: 20 @ 15'-20'
Type: Bulk
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

RESULTS:

Maximum Dry Density (pcf) = 111.6

Optimum Moisture Content (%) = 16.5



* Assumed Gs = 2.65

Project No.: 93-1033

MAXIMUM DENSITY-OPTIMUM MOISTURE

(ASTM D698, METHOD A)

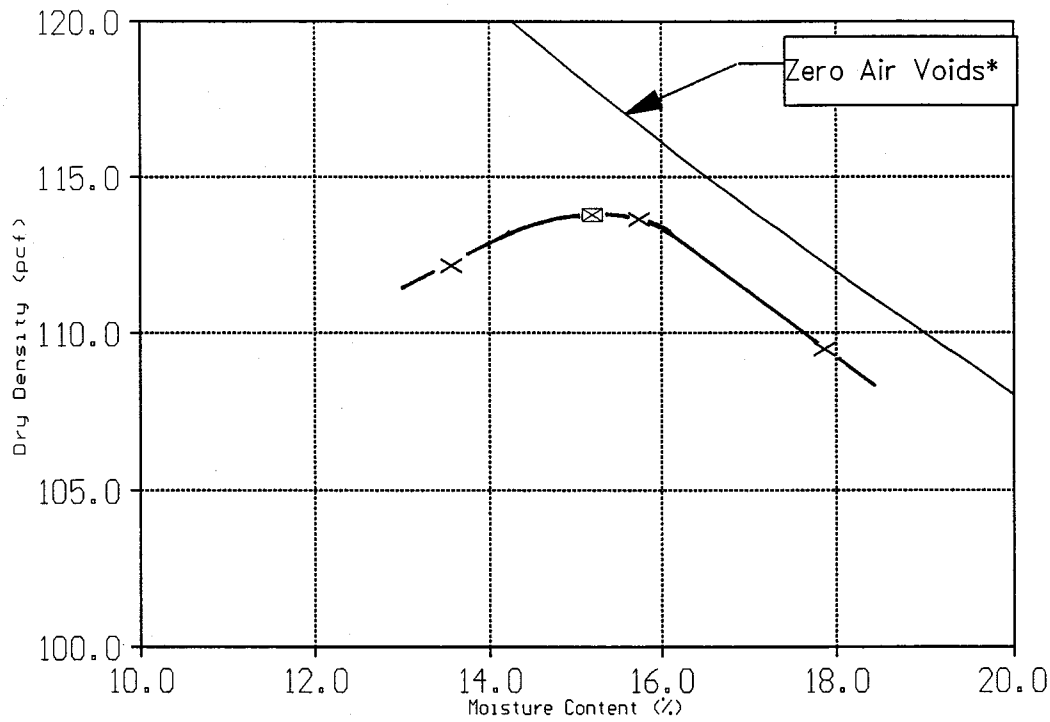
SAMPLE: **Date:** 13-Dec-93

Source: Composite: 21 @ 25'-30' & 21 @ 30'-35'
Type: Bulk
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

RESULTS:

Maximum Dry Density (pcf) = 113.8

Optimum Moisture Content (%) = 15.2



* Assumed Gs = 2.65

Project No.: 93-1033

Thomas-Hartig and Associates, Inc.

MAXIMUM DENSITY-OPTIMUM MOISTURE

(ASTM D698, METHOD A)

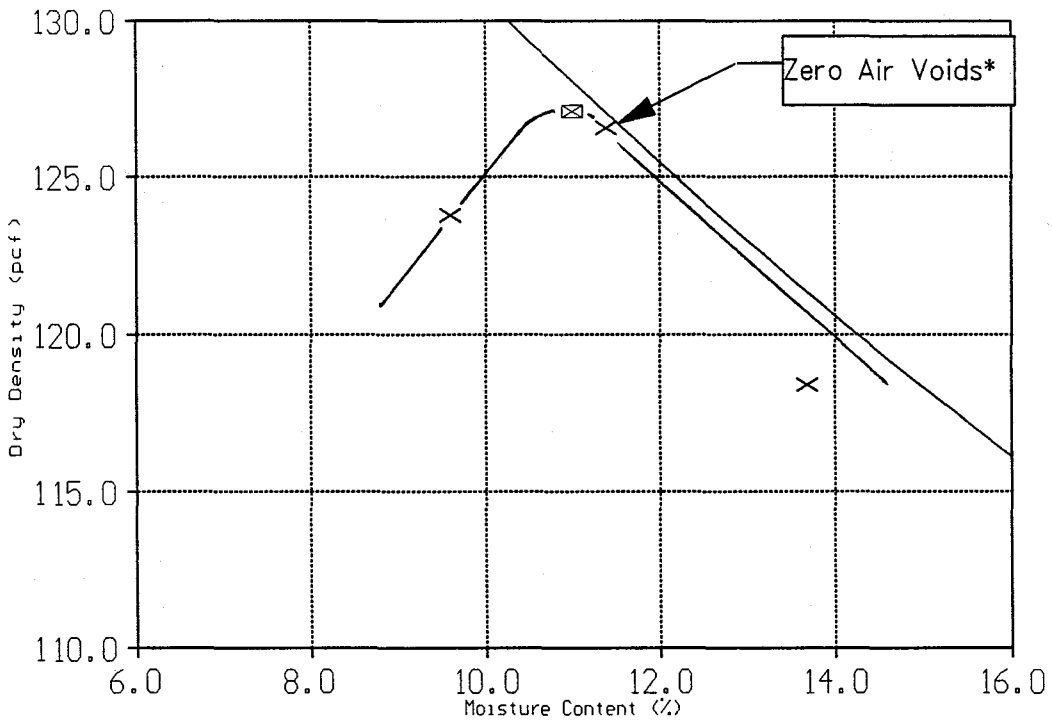
SAMPLE: **Date:** 25-Jan-94

Source: B2 @ 5'-10'
Type: Bulk
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

RESULTS:

Maximum Dry Density (pcf) = 127.1

Optimum Moisture Content (%) = 11.0



* Assumed Gs = 2.65

Project No.: 93-1033

MAXIMUM DENSITY-OPTIMUM MOISTURE

(ASTM D698, METHOD A)

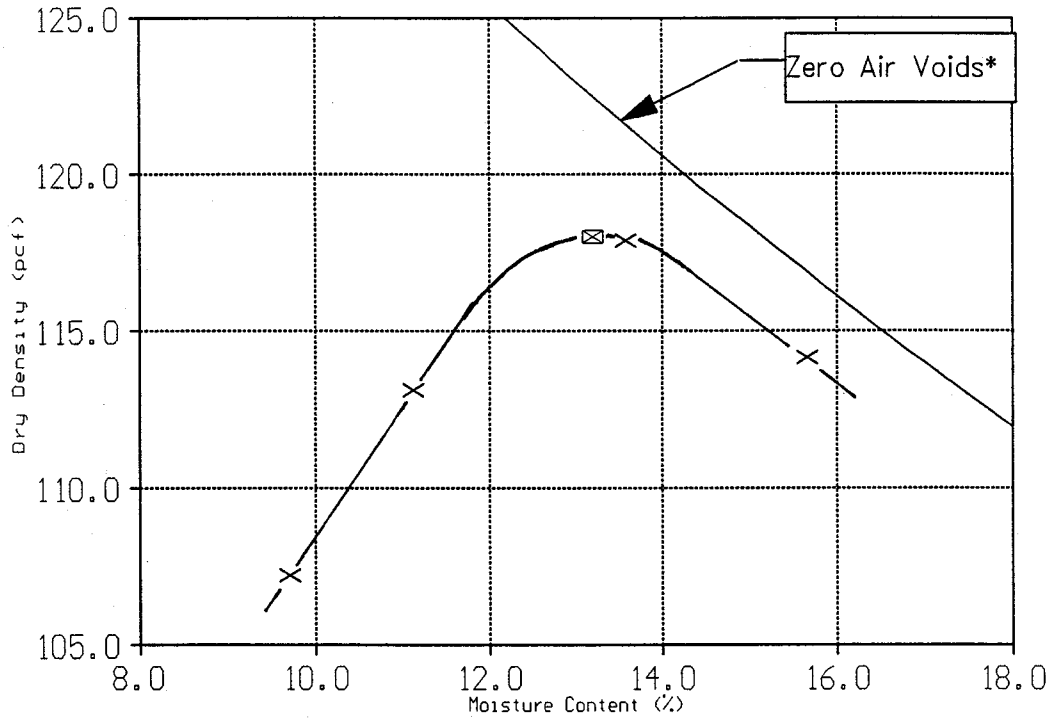
SAMPLE: **Date:** 03-Jan-94

Source: B @ 0'-5'
Type: Bulk
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

RESULTS:

Maximum Dry Density (pcf) = 118.0

Optimum Moisture Content (%) = 13.2



* Assumed Gs = 2.65

Project No.: 93-1033

Thomas-Hartig and Associates, Inc.

MAXIMUM DENSITY-OPTIMUM MOISTURE

(ASTM D698, METHOD A)

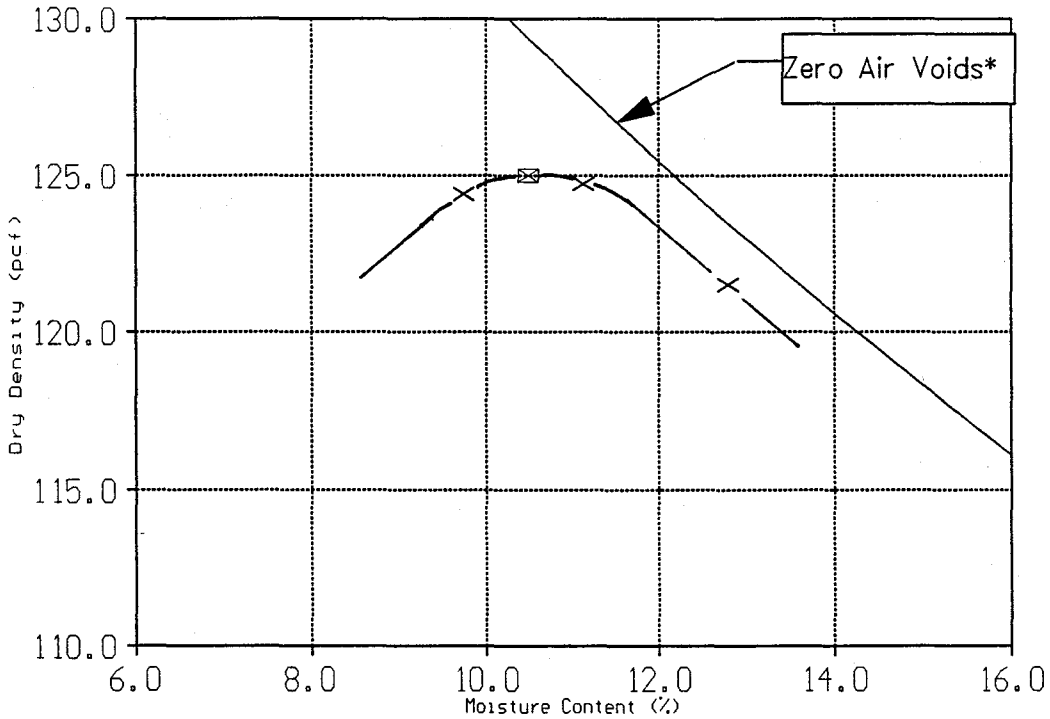
SAMPLE: **Date:** 25-Jan-94

Source: E @ 0'-5'
Type: Bulk
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

RESULTS:

Maximum Dry Density (pcf) = 125.0

Optimum Moisture Content (%) = 10.5



* Assumed Gs = 2.65

Project No.: 93-1033

MAXIMUM DENSITY-OPTIMUM MOISTURE

(ASTM D698, METHOD A)

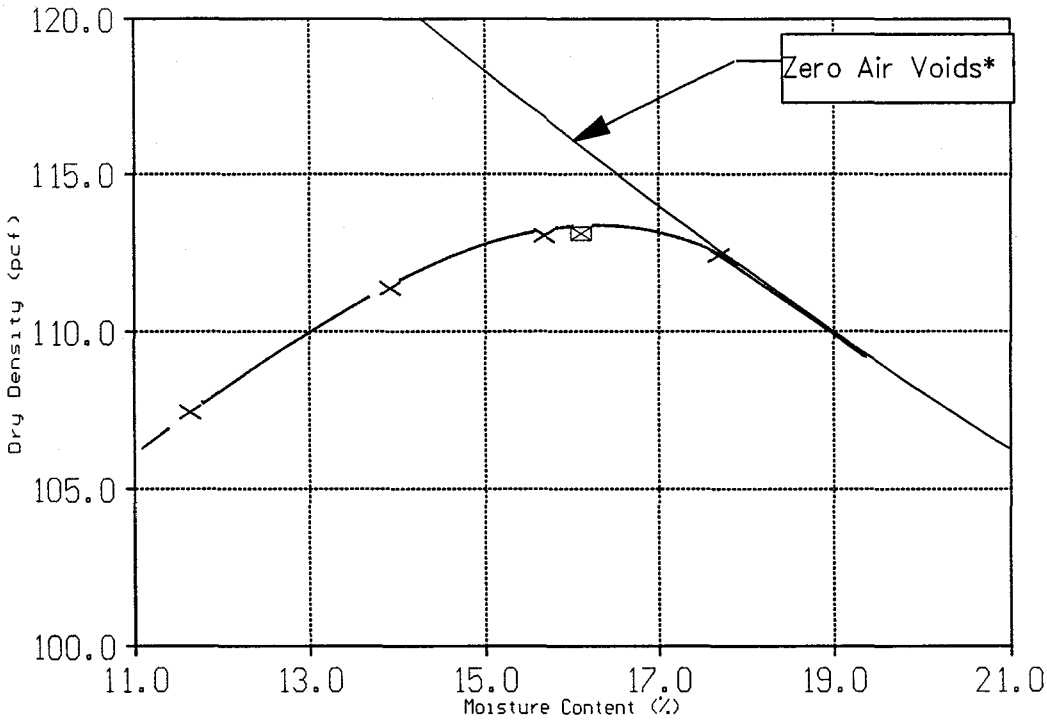
SAMPLE: **Date:** 27-Jan-94

Source: F @ 0'-5'
Type: Bulk
Material: Sandy Clay (CL)
Sampled By: TH/Thompson

RESULTS:

Maximum Dry Density (pcf) = 113.1

Optimum Moisture Content (%) = 16.1



* Assumed Gs = 2.65

Project No.: 93-1033

REPORT ON HVEEM R-VALUE TESTS

Date: 1-24-94

Description:

Source: As Noted Below
Type: Bulk Samples
Material: Surface Soil
Sampled by: TH/Walsh

TESTED: R-Value

RESULTS:

Location	*R-Value
A; 0 - 5'	53
C; 0 - 5'	33
D; 0 - 5'	27
G; 0 - 5;	64

*corrected to 300 psi exudation pressure

Project No. 93-1033

Thomas-Hartig & Associates, Inc.

REPORT ON EXPANSION TEST

SAMPLE: Date: 1-28-94

Source: Noted Below
Type: Grab Sample
Material: Subsurface Soil
Sampled By: TH/Thompson

TESTED: [1]Percent expansion upon soaking of remolded sample compacted to approximately 95% of the maximum ASTM D698 dry density at approximately 2% less than optimum moisture content. [2]Percent expansion upon soaking of undisturbed sample.

TEST RESULTS

Sample	Dry Density (pcf)	Initial Moisture (Percent)	Surcharge Pressure (psf)	Expansion Upon Soaking (Percent)
B; 0 - 5'	113	11.2	100	1.80[1]
5; 0 - 2'	109	13.1	100	1.30[1]
10; 3 - 8'	110	12.8	100	2.45[1]
15; 10 - 15'	99	18.8	100	6.05[1]
19; 15 - 20'	92	23.6	100	16.00[1]
19; 14 - 15'	93	29.9	100	12.93[2]
20; 15 - 20'	106	14.5	100	7.92[1]
21; 25 - 35'	108	13.2	100	6.71[1]
R-4; 6 - 12'	107	14.0	100	1.53[1]
B-2; 5 - 10'	121	9.0	100	0.52[2]
B-5; 10 - 15'	114	10.7	100	0.48[1]

Project No. 93-1033

Thomas-Hartig & Associates, Inc.

APPENDIX C

CALCULATIONS

93-1033
By: KW 1/24/94
Checked KEP 1/26/94

PERCOLATION TEST RESULTS

P-1 CLAY

① Time	② Depth to water (in)	③ (from ①) Minutes since last measure	④ (from ②) Drop in water surface (in)	⑤ = ③/④ Percolation rate (min/in)
8:52AM	30	FILL	—	—
10:28AM	68	96	38	2.5
12:31PM	135	123	67	1.8

P-2 SAND

①	②	③	④	⑤
8:34AM	65	FILL	—	—
9:03AM	116	29	51	0.57
9:04AM	91	FILL	—	—
10:37AM	144	93	53	1.8
12:38PM	EMPTY	119	(MIN. 24)	(MAX 50)

P-3 CLAY - SANDY

①	②	③	④	⑤
7:53AM	60	FILL	—	—
8:55AM	108	62	48	1.3
8:56AM	89	FILL	—	—
10:32AM	136	96	47	2.0
12:35AM	EMPTY	123	(MIN. 56)	(MAX 2.2)

by: KW 93-1033

12/13/93

Checked KER 12/26/93

SLOPE STABILITY ANALYSES

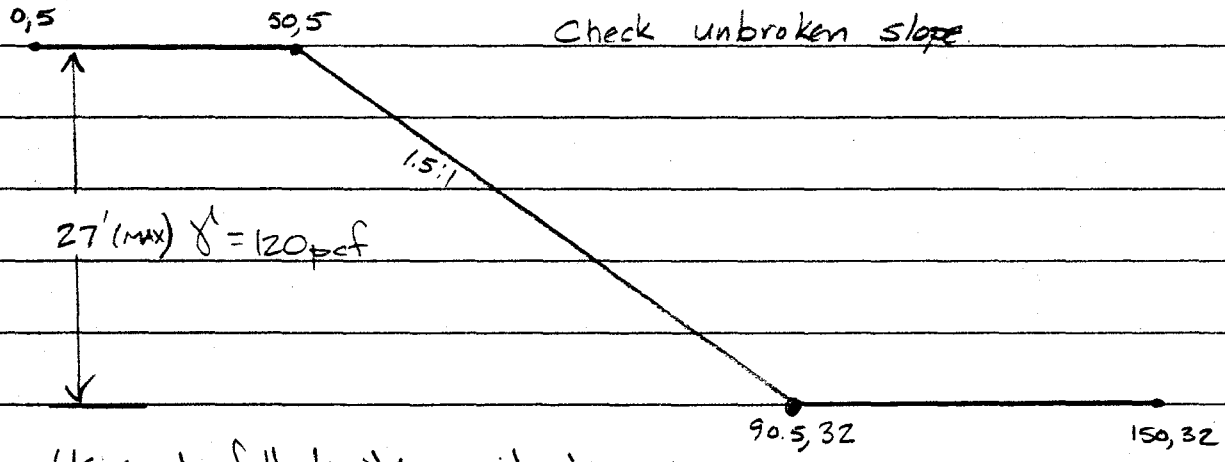
From Culman's analysis:

for FS = 1 : vertical slope,

$$H = \frac{4c}{\gamma} = \frac{4(650\,psf)}{120\,pcf} = 21.7'$$

We are designing at 27' max. 1.5:1 much less critical than vertical.

using $c = 200°$ $\therefore$ $\underline{\underline{H}} = \frac{4(200)}{120} = \underline{\underline{6.7'}}$



Using 1 full depth soil type

	$\phi = 32°$ $c = 650$		$\phi = 20°$ $c = 850$		$\phi = 30°$ $c = 300$		$\phi = 30°$ $c = 200$	
Tangent depth	Center	FS	Ctr	FS	Ctr	FS	Ctr	FS
25					77, -13	2.19	79, -14	1.84
30	81, -16	3.01	78, -16	2.92	84, -16	$\boxed{2.00}$	86, -17	$\boxed{1.70^*}$
35	79, -7	$\boxed{2.94}$	76, -7	$\boxed{2.69}$	82, -7	2.05	84, -8	1.81
40	79, -5	3.19	76, -6	2.79			Results attached	
45	79, -5	3.51						
50	80, -7	3.87						

REPORT ON LABORATORY TESTS

KW

Date 12/10/93 - 1/25/94

Checked 1/26/94

SAMPLE:

Source Noted below

Type Driven Ring Samples

Material SAND SAMPLES

Sampled By

TESTED: Direct shear results- plotted together.

RESULTS:

Friction Angle (Ø) =

Cohesion (c) =

R3; 9 - 10' (SM) △

4; 4 - 5' (SM-GM) □

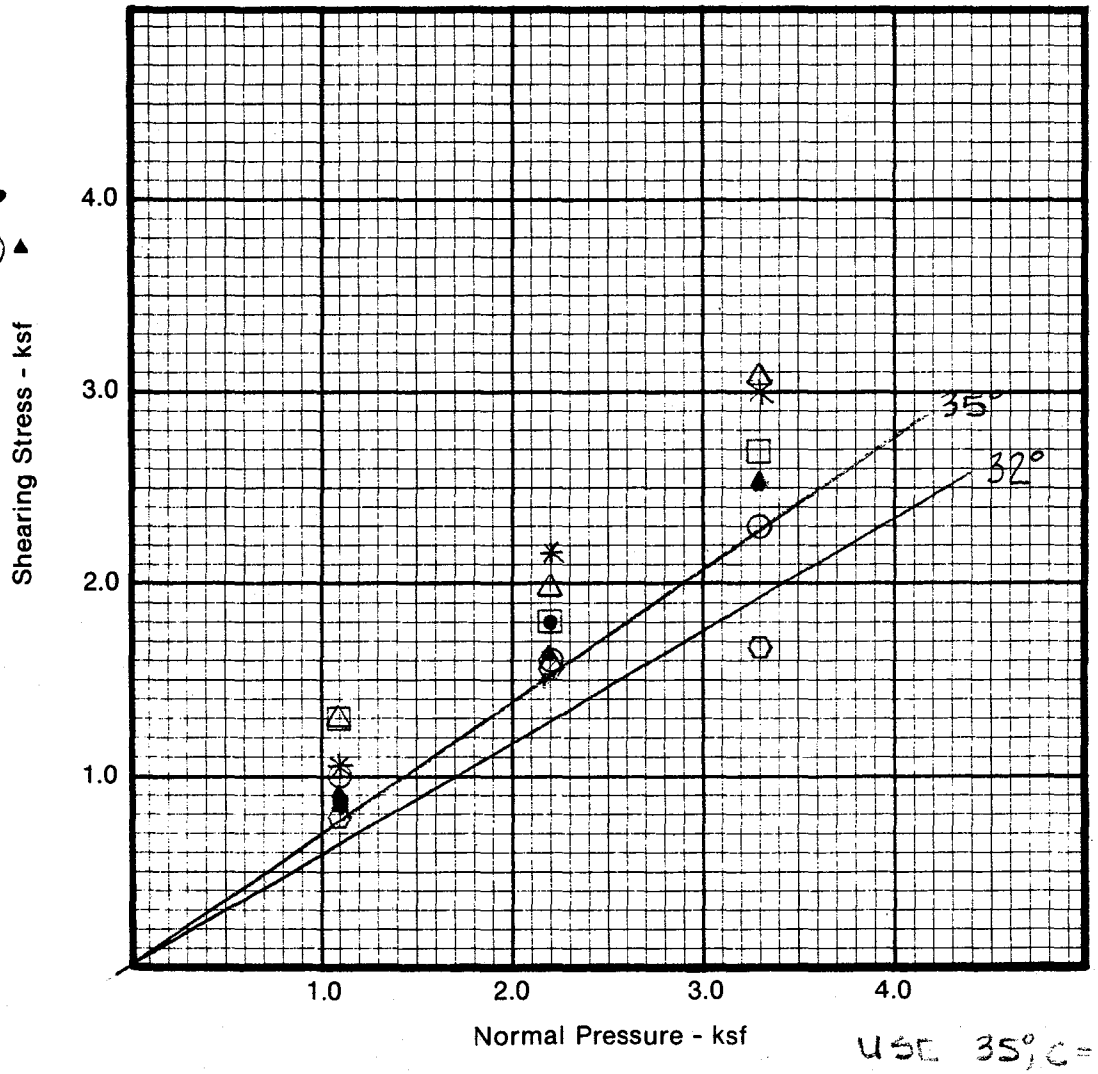
11; 9.5 - 10.5 (SM) ○

23; 14 - 15' (SM) ✱

25; 9 - 10' (SP-SM) ●

6; 19-20' (SW-GW) ▲

0; 50 - 51' (SM) ⬡



USE 35°, c = 0

Project No. 93-1033

TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

REPORT ON LABORATORY TESTS

KW

Date 12/10/93 - 1/25/94

Collection ~~[illegible]~~ 1/26/94

SAMPLE:

Source Noted below

Type Driven ring samples

Material CLAY SAMPLES

Sampled By

TESTED: Direct shear results - plotted together

RESULTS:

Friction Angle (Ø) =

Cohesion (c) =

Speedie; 1.5' & 2.5' • (CL, CL-ML)

1; 4-5' (CL) △

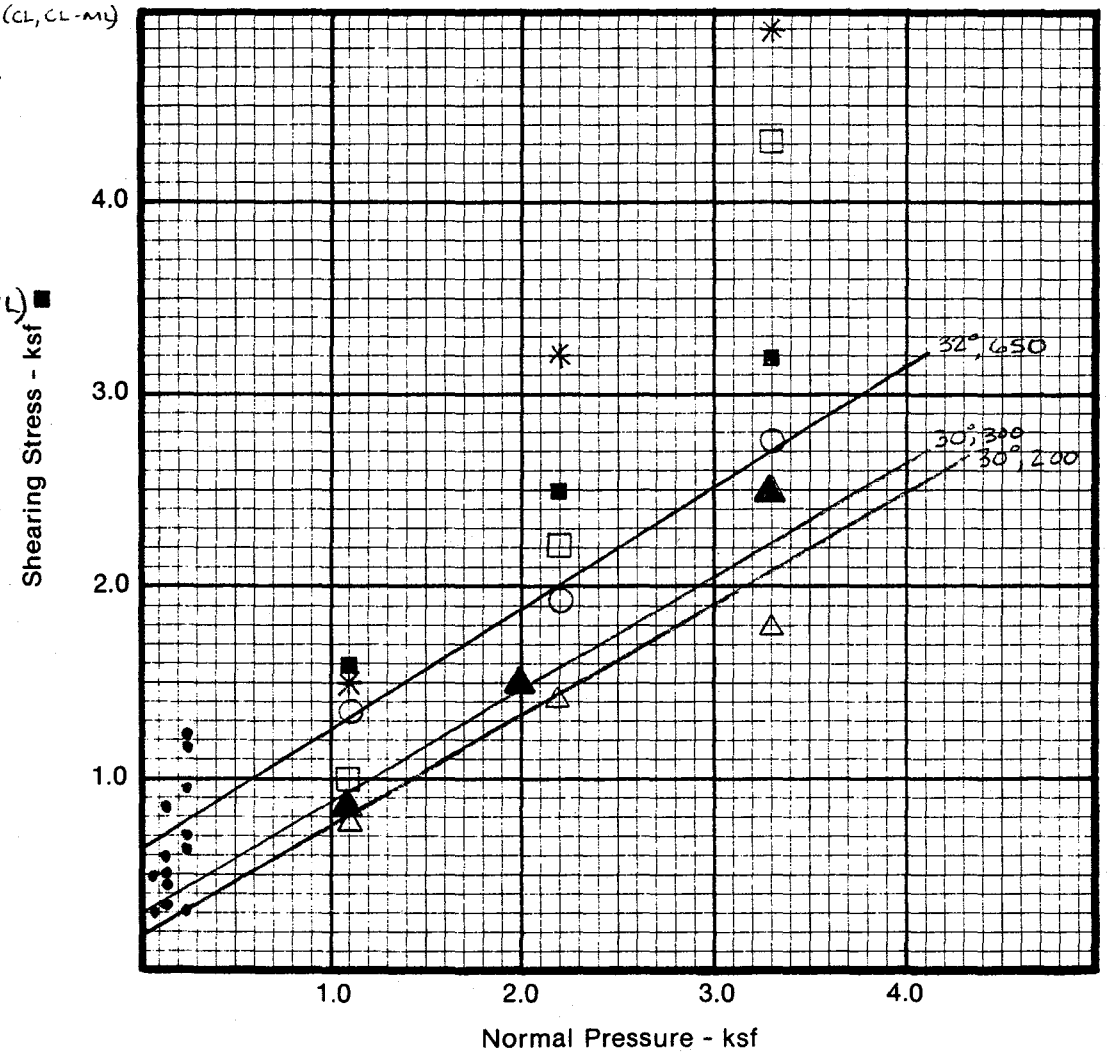
17; 4-5' (CL) □

18; 19-20' (MH) ○

B10; 20-21' (CH) ✳

B16; 10-11' (CL) ▲

B18; 35-36' (SC-CL) ■



USE 30°, 200 PSF

Project No.

TH-105

THOMAS-HARTIG & ASSOCIATES, INC.

by: KW 93-1033
2/13/93

1033.DAT

CHECKED EK 1/26/94

```
PRELIMINARY CASE -- DYSART DRAIN EVALUATION AT 1.5:1
2 0 1 4 2 0 0 0 0 0 0 0
3 0 0 0 0 0 2116 62.4 0 0
0 0 10 0 0
85 -15 1
30
0     5  5  5  150
50    5  5  5  150
90.5 32 32 32 150
150  32 32 32 150
1 120 200 30 0
STOP
```

by: KW 93-1033
12/13/93
Checked KR 1-28-94

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* ********************************************* *
* *                                         * *
* *    --------------------------------     * *
* *     TSTAB  slope stability analysis     * *
* *        Revision 2.40 - 02/12/85         * *
* *    --------------------------------     * *
* *                                         * *
* *    TAGA Engineering Software Services   * *
* *           Berkeley, California          * *
* *                                         * *
* *   IBM PC & 8086/8088 MS-DOS Version by  * *
* *                                         * *
* * Design Professionals Management Systems * *
* *           Kirkland, Washington          * *
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* *                                         * *
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*********************************************************************
  PRELIMINARY CASE -- DYSART DRAIN EVALUATION AT 1.5:1
*********************************************************************
```

```
*********************************
  ANALYSIS BY SPENCER'S METHOD
*********************************
```

```
**************
  INPUT DATA
**************
```

```
 CONTROL DATA,
      AUTOMATIC SEARCH FOR CRITICAL CIRCLE
      NUMBER OF DEPTH LIMITING TANGENTS                1
      NUMBER OF VERTICAL SECTIONS                      4
      NUMBER OF SOIL LAYER BOUNDARIES                  2
      NUMBER OF POINTS DEFINING COHESION PROFILE       0
      NUMBER OF CURVES DEFINING COHESION ANISOTROPY    0
      NUMBER OF BOUNDARY LINE LOADS                    0
      NUMBER OF BOUNDARY PRESSURE LOADS                0

      SEISMIC COEFFICIENT                    =       .000
      ATMOSPHERIC PRESSURE                   =   2116.000
      UNIT WEIGHT OF WATER                   =     62.400
      UNIT WEIGHT OF WATER IN TENSION CRACK  =     62.400
      INITIAL ESTIMATE OF F                  =      3.000
      INITIAL ESTIMATE OF THETA              =     10.000
      ALLOWABLE FORCE IMBALANCE              =     10.000
      ALLOWABLE MOMENT IMBALANCE             =    100.000

 SEARCH STARTS AT CENTER (  85.0, -15.0),WITH FINAL GRID OF  1.0

 ALL CIRCLES TANGENT TO DEPTH,  30.0,

 GEOMETRY

     SECTIONS        .00    50.00    90.50   150.00

     T. CRACKS      5.00     5.00    32.00    32.00
     W IN CRACK     5.00     5.00    32.00    32.00
     BOUNDARY 1     5.00     5.00    32.00    32.00
```

by: KW 93-1033
12/13/93
Checked KK
1-26-94

BOUNDARY 2 150.00 150.00 150.00 150.00

SOIL PROPERTIES

LAYER	DENSITY	COHESION	FRICTION ANGLE	DELTA PHI
1	120.00	200.00	30.00	.00

1

****
RESULTS
****

****
DEPTH LIMITING TANGENT NO. 1 AT Y = 30.00
****

NUMBER	TANGENT	RADIUS	(X) CENTER	(Y) CENTER	F.S.
1	30.0	45.0	85.0	-15.0	1.701
2	30.0	45.0	83.0	-15.0	1.718
3	30.0	47.0	85.0	-17.0	1.703
4	30.0	45.0	87.0	-15.0	1.710
5	30.0	43.0	85.0	-13.0	1.705
6	30.0	45.0	84.0	-15.0	1.707
7	30.0	46.0	85.0	-16.0	1.701
8	30.0	45.0	86.0	-15.0	1.701
9	30.0	44.0	85.0	-14.0	1.702
10	30.0	46.0	84.0	-16.0	1.709
11	30.0	46.0	86.0	-16.0	1.699
12	30.0	44.0	86.0	-14.0	1.705
13	30.0	44.0	84.0	-14.0	1.706
14	30.0	47.0	86.0	-17.0	1.698
15	30.0	46.0	87.0	-16.0	1.704
16	30.0	47.0	85.0	-17.0	1.703
17	30.0	48.0	86.0	-18.0	1.699
18	30.0	47.0	87.0	-17.0	1.700
19	30.0	48.0	85.0	-18.0	1.705
20	30.0	48.0	87.0	-18.0	1.698
21	30.0	46.0	87.0	-16.0	1.704
22	30.0	46.0	85.0	-16.0	1.701

F.S. MINIMUM= 1.698 FOR THE CIRCLE OF CENTER (86.0, -17.0)

****
DETAIL OF CALCULATION FOR CRITICAL CIRCLE
****

ITERATION	F	THETA DEGREES	EXCESS FORCE	EXCESS MOMENT
1	3.000	10.0	-8421.8	65721.5
2	2.000	8.6	-3335.6	12692.5
3	1.622	-1.4	-303.7	-38856.3
4	1.651	8.6	-215.2	-26853.9
5	1.679	18.6	-98.8	-12338.1
6	1.698	25.4	6.4	216.5
7	1.698	25.3	.0	.0

1

****
LOCATION OF CRITICAL CIRCLE
****

++ +

by: KW 93-1083
12/13/93

```
  +* +
 + ++ +
+ + ++ +
 + ++
  +
```

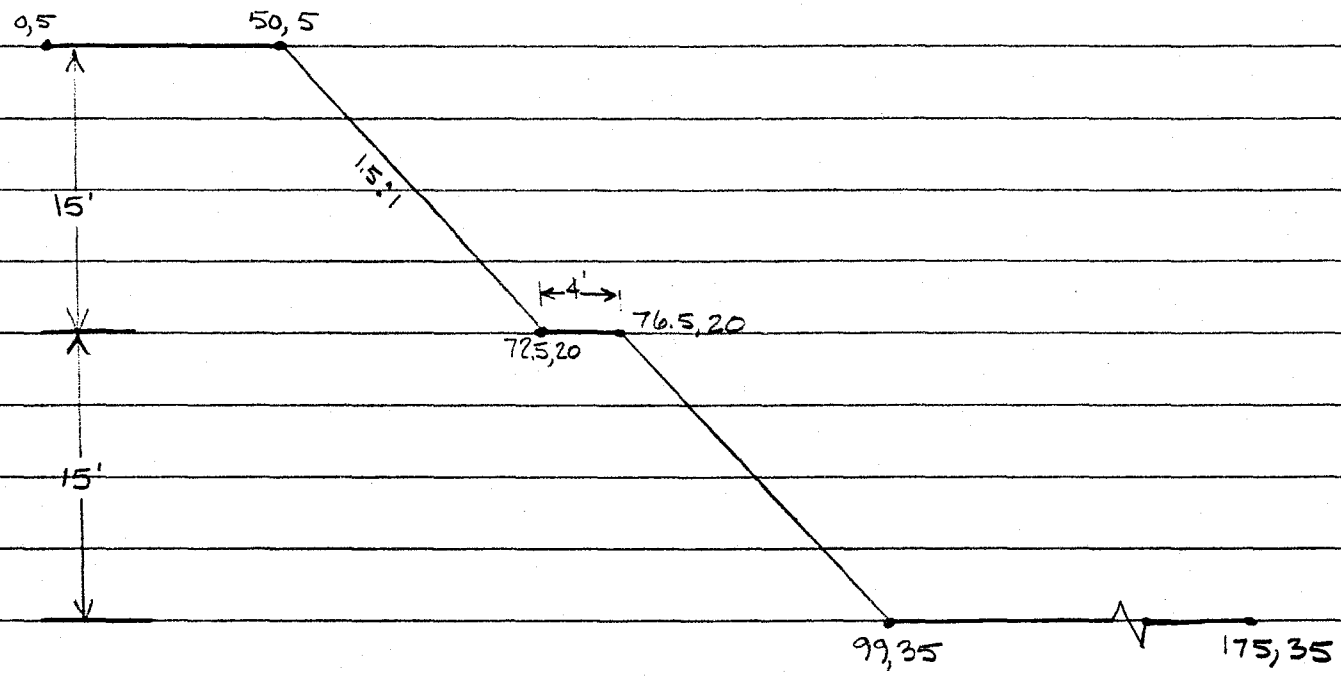
```
OOOOOOOOOOOOOOOOOOOOOOOOOO
                OXXXXXXXXXOOO
                 XXXXXXXXXXXOO
                 OOXXXXXXXXXXXOOO
                   OXXXXXXXXXXXXXOO
                    XXXXXXXXXXXXXXXOOO
                    OOXXXXXXXXXXXXXXXXXOO
                      OXXXXXXXXXXXXXXXXXXXOOO
                       OOXXXXXXXXXXXXXXXXXXXXOO
                         OXXXXXXXXXXXXXXXXXXXXXOOO
                          OXXXXXXXXXXXXXXXXXXXXXXOO
                           OOXXXXXXXXXXXXXXXXXXXXXXXOOO
                             OOOXXXXXXXXXXXXXXXXXXXXXXXXOO
                                OOXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                  OOXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                    OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                      OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                        OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                          OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                            OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                              OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                  OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                    OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                      OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                        OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                          OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                            OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                              OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                                OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                                  OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                                    OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                                      OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                                        OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                                          OOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                                            OOOOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                                                OOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                                                  OOOOOOOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                                                        OOOOOOOOOXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                                                                OOOOOOOOOOOOOOOOOOOO
```

by: KW 93-1033

1/25/94

Checked [illegible] 1-26-94

Check benched slope; 30' deep per meeting @ NBS/Lowry



$\phi = 30°$ $c = 200$

Tangent depth	Ctr	FS
30	89,-25	1.86
32.5	93,-27	1.79
35	98,-31	$\boxed{1.73}$*
37.5	91,-15	1.83
40	90,-12	1.94

* results attached

DW LW 93-1033
1/25/94

1033 BNCH.DAT

PRELIMINARY CASE -- DYSART DRAIN EVALUATION AT 1.5:1 -- BENCHED
2 0 1 6 2 0 0 0 0 0 0
3 0 0 0 0 0 2116 62.4 0 0
0 0 10 0 0
95 -30 1
35
0 5 5 5 150
50 5 5 5 150
72.5 20 20 20 150
76.5 20 20 20 150
99 35 35 35 150
175 35 35 35 150
1 120 200 30 0
STOP

by: KW 93-1033
/25/94
1 OF 3
Checked [illegible]

```
************************************************
* ******************************************** *
* *                                        * *
* *    --------------------------------    * *
* *     TSTAB  slope stability analysis    * *
* *        Revision 2.40 - 02/12/85        * *
* *    --------------------------------    * *
* *                                        * *
* *    TAGA Engineering Software Services  * *
* *           Berkeley, California         * *
* *                                        * *
* *   IBM PC & 8086/8088 MS-DOS Version by * *
* *                                        * *
* * Design Professionals Management Systems * *
* *           Kirkland, Washington         * *
* *                                        * *
* *      copyright (c) 1983, 1984 TAGA     * *
* *      copyright (c) 1983, 1984 DPMS     * *
* *                                        * *
* ******************************************** *
************************************************
```

```
*********************************************************************
  PRELIMINARY CASE -- DYSART DRAIN EVALUATION AT 1.5:1 -- BENCHED
*********************************************************************
```

```
*********************************
  ANALYSIS BY SPENCER'S METHOD
*********************************
```

```
**************
  INPUT DATA
**************
```

```
 CONTROL DATA,
      AUTOMATIC SEARCH FOR CRITICAL CIRCLE
      NUMBER OF DEPTH LIMITING TANGENTS                1
      NUMBER OF VERTICAL SECTIONS                      6
      NUMBER OF SOIL LAYER BOUNDARIES                  2
      NUMBER OF POINTS DEFINING COHESION PROFILE       0
      NUMBER OF CURVES DEFINING COHESION ANISOTROPY    0
      NUMBER OF BOUNDARY LINE LOADS                    0
      NUMBER OF BOUNDARY PRESSURE LOADS                0

      SEISMIC COEFFICIENT                    =       .000
      ATMOSPHERIC PRESSURE                   =   2116.000
      UNIT WEIGHT OF WATER                   =     62.400
      UNIT WEIGHT OF WATER IN TENSION CRACK  =     62.400
      INITIAL ESTIMATE OF F                  =      3.000
      INITIAL ESTIMATE OF THETA              =     10.000
      ALLOWABLE FORCE IMBALANCE              =     10.000
      ALLOWABLE MOMENT IMBALANCE             =    100.000

 SEARCH STARTS AT CENTER (  95.0, -30.0),WITH FINAL GRID OF  1.0

 ALL CIRCLES TANGENT TO DEPTH,  35.0,

 GEOMETRY

     SECTIONS        .00    50.00    72.50    76.50    99.00   175.00

     T. CRACKS      5.00     5.00    20.00    20.00    35.00    35.00
     W IN CRACK     5.00     5.00    20.00    20.00    35.00    35.00
     BOUNDARY 1     5.00     5.00    20.00    20.00    35.00    35.00
```

by KW 93-1033
1/25/94
2 OF 3
[illegible] 1-26-94

BOUNDARY 2 150.00 150.00 150.00 150.00 150.00 150.00

SOIL PROPERTIES

LAYER	DENSITY	COHESION	FRICTION ANGLE	DELTA PHI
1	120.00	200.00	30.00	.00

1

****
RESULTS
****

****
DEPTH LIMITING TANGENT NO. 1 AT Y = 35.00
****

NUMBER	TANGENT	RADIUS	(X) CENTER	(Y) CENTER	F.S.
1	35.0	65.0	95.0	-30.0	1.743
2	35.0	65.0	93.0	-30.0	1.768
3	35.0	67.0	95.0	-32.0	1.750
4	35.0	65.0	97.0	-30.0	1.730
5	35.0	63.0	95.0	-28.0	1.738
6	35.0	65.0	96.0	-30.0	1.735
7	35.0	66.0	97.0	-31.0	1.731
8	35.0	65.0	98.0	-30.0	1.728
9	35.0	64.0	97.0	-29.0	1.729
10	35.0	66.0	98.0	-31.0	1.728
11	35.0	65.0	99.0	-30.0	1.731
12	35.0	64.0	98.0	-29.0	1.729
13	35.0	66.0	97.0	-31.0	1.731
14	35.0	67.0	98.0	-32.0	1.729
15	35.0	66.0	99.0	-31.0	1.730
16	35.0	67.0	97.0	-32.0	1.733
17	35.0	67.0	99.0	-32.0	1.729
18	35.0	65.0	99.0	-30.0	1.731
19	35.0	65.0	97.0	-30.0	1.730

F.S. MINIMUM= 1.728 FOR THE CIRCLE OF CENTER (98.0, -31.0)

****
DETAIL OF CALCULATION FOR CRITICAL CIRCLE
****

ITERATION	F	THETA DEGREES	EXCESS FORCE	EXCESS MOMENT
1	3.000	10.0	-11366.0	107024.6
2	2.000	8.2	-4206.6	13446.0
3	1.798	-1.8	-2414.2	-17994.3
4	1.836	8.2	-2311.1	-16707.7
5	1.871	18.2	-2148.7	-14440.1
6	1.884	28.2	-1702.7	-5713.6
7	1.711	18.2	-110.4	-21443.1
8	1.728	24.6	7.4	263.0
9	1.728	24.5	.0	.0

1

****
LOCATION OF CRITICAL CIRCLE
****

+ ++ +
+* +

by: KW 93-1033
1/25/94
3 of 3
Checked 1-26-94
KK

```
+  ++++ +
   +
```

```
OOOOOOOOOOOOOOOOOOOOOOOOOOO
                OXXXXXXXXXXXOO
                 OOXXXXXXXXXXXOO
                   OXXXXXXXXXXXXOOO
                    OXXXXXXXXXXXXXXOO
                     OXXXXXXXXXXXXXXXOO
                      OOXXXXXXXXXXXXXXXOOO
                        OOOXXXXXXXXXXXXXXXOO
                           OXXXXXXXXXXXXXXXXOO
                            OXXXXXXXXXXXXXXXXXOOO
                             OOXXXXXXXXXXXXXXXXXXOO
                               OOXXXXXXXXXXXXXXXXXXOO
                                 OOXXXXXXXXXXXXXXXXXXXOOOOOOOO
                                   OOXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                     OOOXXXXXXXXXXXXXXXXXXXXXXXXXOO
                                        OOXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                          OOXXXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                            OOOOOXXXXXXXXXXXXXXXXXXXXXXXXOO
                                                 OOOXXXXXXXXXXXXXXXXXXXXXXXXOOO
                                                    OOOOXXXXXXXXXXXXXXXXXXXXXXOOO
                                                        OOOOOXXXXXXXXXXXXXXXXXXXXXOO
                                                             OOOOOXXXXXXXXXXXXXXXXXXOOO
                                                                  OOOOOOOOOXXXXXXXXXXOO
                                                                           OOOOOOOOOOOOOOOOOOOOOOOOOOO
```

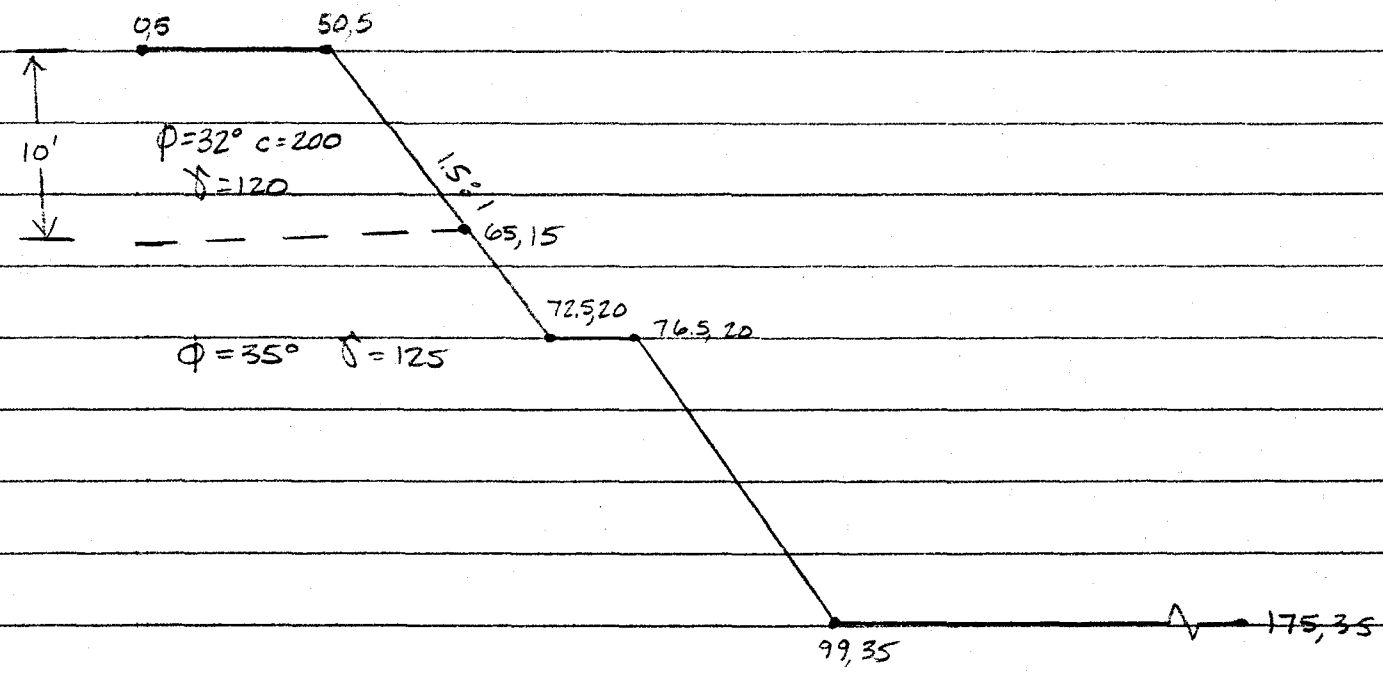
```
-----------------------------------------
Execution complete, time =     4.51 seconds
-----------------------------------------
```

by: KW 93-1033

1/25/94

Checked 1-26-94

Check with 10' clay over sand



DEPTH	CTR, FS		
30	111,-35	1.18	(∞ SLOPE → CARRY LINER TO TOP OF SAND)
35	106,-37	1.45	OK RESULTS ATTACHED
40	95,-14	1.03	

by: KW 93-1033
1-25-94
Checked [illegible] 1-26-94

1033SAND.DAT

DOWNSTREAM CASE -- DYSART DRAIN EVALUATION AT 1.5:1 -- BENCHED
2 0 1 7 3 0 0 0 0 0 0 0
3 0 0 0 0 0 2116 62.4 0 0
0 0 10 0 0
105 -35 1
35
0 5 5 5 15 150
50 5 5 5 15 150
65 15 15 15 15 150
72.5 20 20 20 20 150
76.5 20 20 20 20 150
99 35 35 35 35 150
175 35 35 35 35 150
1 120 200 30 0
2 125 0.1 35 0
STOP

by: KW 93-1033
1/25/94
Checked [illegible] 1-27-94
1 of 3

```
*************************************************
* ********************************************* *
* *                                           * *
* *    ---------------------------------      * *
* *     TSTAB  slope stability analysis       * *
* *        Revision 2.40 - 02/12/85           * *
* *    ---------------------------------      * *
* *                                           * *
* *    TAGA Engineering Software Services     * *
* *           Berkeley, California            * *
* *                                           * *
* *   IBM PC & 8086/8088 MS-DOS Version by    * *
* *                                           * *
* * Design Professionals Management Systems   * *
* *           Kirkland, Washington            * *
* *                                           * *
* *      copyright (c) 1983, 1984 TAGA        * *
* *      copyright (c) 1983, 1984 DPMS        * *
* *                                           * *
* ********************************************* *
*************************************************
```

```
*************************************************************************
  DOWNSTREAM CASE -- DYSART DRAIN EVALUATION AT 1.5:1 -- BENCHED
*************************************************************************
```

```
*********************************
  ANALYSIS BY SPENCER'S METHOD
*********************************
```

```
**************
  INPUT DATA
**************
```

```
 CONTROL DATA,
      AUTOMATIC SEARCH FOR CRITICAL CIRCLE
      NUMBER OF DEPTH LIMITING TANGENTS                 1
      NUMBER OF VERTICAL SECTIONS                       7
      NUMBER OF SOIL LAYER BOUNDARIES                   3
      NUMBER OF POINTS DEFINING COHESION PROFILE        0
      NUMBER OF CURVES DEFINING COHESION ANISOTROPY     0
      NUMBER OF BOUNDARY LINE LOADS                     0
      NUMBER OF BOUNDARY PRESSURE LOADS                 0

      SEISMIC COEFFICIENT                    =       .000
      ATMOSPHERIC PRESSURE                   =   2116.000
      UNIT WEIGHT OF WATER                   =     62.400
      UNIT WEIGHT OF WATER IN TENSION CRACK  =     62.400
      INITIAL ESTIMATE OF F                  =      3.000
      INITIAL ESTIMATE OF THETA              =     10.000
      ALLOWABLE FORCE IMBALANCE              =     10.000
      ALLOWABLE MOMENT IMBALANCE             =    100.000

 SEARCH STARTS AT CENTER ( 105.0, -35.0),WITH FINAL GRID OF  1.0

 ALL CIRCLES TANGENT TO DEPTH,  35.0,

 GEOMETRY

     SECTIONS        .00    50.00    65.00    72.50    76.50    99.00   175.00

     T. CRACKS      5.00     5.00    15.00    20.00    20.00    35.00    35.00
     W IN CRACK     5.00     5.00    15.00    20.00    20.00    35.00    35.00
     BOUNDARY 1     5.00     5.00    15.00    20.00    20.00    35.00    35.00
```

by: KW 1/25/94
Checked 1-27-94

2 OF 3

BOUNDARY 2	15.00	15.00	15.00	20.00	20.00	35.00	35.00
BOUNDARY 3	150.00	150.00	150.00	150.00	150.00	150.00	150.00

SOIL PROPERTIES

LAYER	DENSITY	COHESION	FRICTION ANGLE	DELTA PHI
1	120.00	200.00	30.00	.00
2	125.00	.10	35.00	.00

1

****
RESULTS
****

****
DEPTH LIMITING TANGENT NO. 1 AT Y = 35.00
****

NUMBER	TANGENT	RADIUS	(X) CENTER	(Y) CENTER	F.S.
1	35.0	70.0	105.0	-35.0	1.450
2	35.0	70.0	103.0	-35.0	1.465
3	35.0	72.0	105.0	-37.0	1.452
4	35.0	70.0	107.0	-35.0	1.461
5	35.0	68.0	105.0	-33.0	1.453
6	35.0	70.0	104.0	-35.0	1.455
7	35.0	71.0	105.0	-36.0	1.450
8	35.0	70.0	106.0	-35.0	1.451
9	35.0	69.0	105.0	-34.0	1.451
10	35.0	71.0	104.0	-36.0	1.457
11	35.0	71.0	106.0	-36.0	1.449
12	35.0	69.0	106.0	-34.0	1.455
13	35.0	69.0	104.0	-34.0	1.454
14	35.0	72.0	106.0	-37.0	1.448
15	35.0	71.0	107.0	-36.0	1.455
16	35.0	72.0	105.0	-37.0	1.452
17	35.0	73.0	106.0	-38.0	1.448
18	35.0	72.0	107.0	-37.0	1.451
19	35.0	73.0	105.0	-38.0	1.454
20	35.0	73.0	107.0	-38.0	1.448
21	35.0	71.0	107.0	-36.0	1.455
22	35.0	71.0	105.0	-36.0	1.450

F.S. MINIMUM= 1.448 FOR THE CIRCLE OF CENTER (106.0, -37.0)

****
DETAIL OF CALCULATION FOR CRITICAL CIRCLE
****

ITERATION	F	THETA DEGREES	EXCESS FORCE	EXCESS MOMENT
1	3.000	10.0	-8845.2	84374.9
2	2.000	9.6	-4680.3	40078.4
3	1.546	19.6	-1066.5	-7368.7
4	1.439	9.6	-156.6	-13713.5
5	1.446	19.6	-79.3	-6309.9
6	1.448	25.5	1.9	107.3
7	1.448	25.4	.0	.0

1

****
LOCATION OF CRITICAL CIRCLE
****

by: KW 93-1033
1/25/94

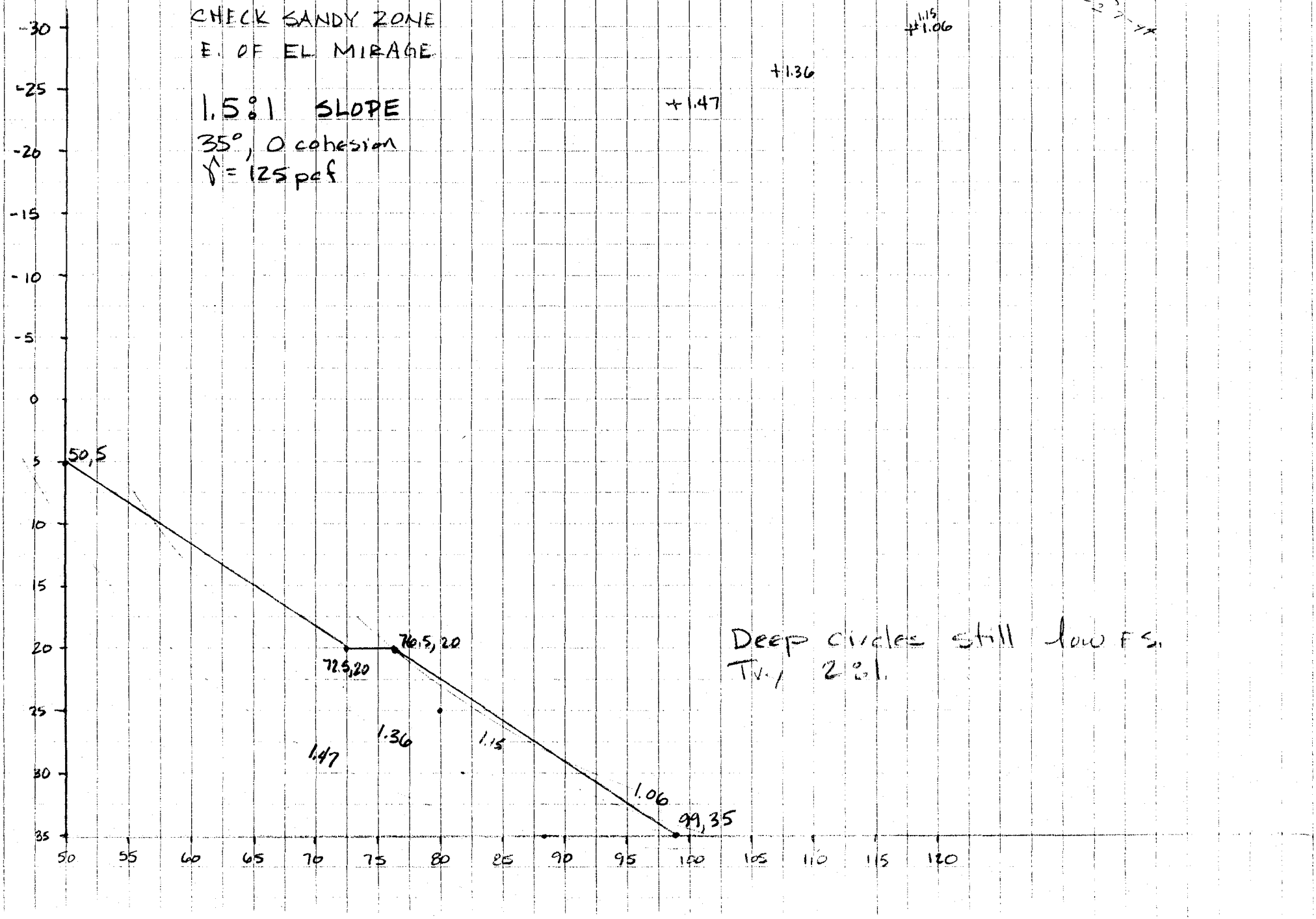
Attached [illegible] 14
3 OF 3

```
      ++ +
      +* +
     +++ +
   + +++ +
      +
```

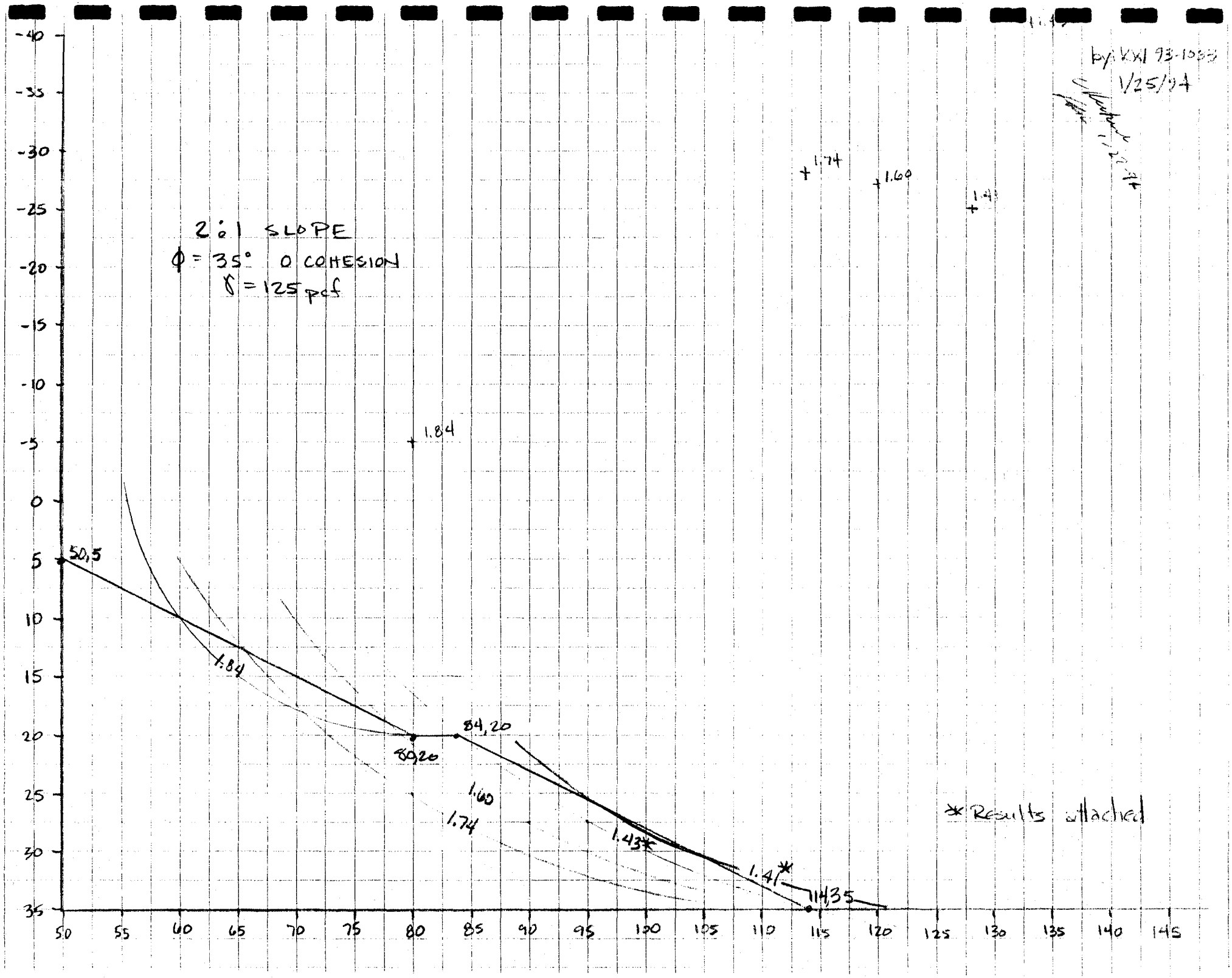
```
OOOOOOOOOOOOOOOOOOOO
                OXXXXXOO
                 OOXXXXXOO
                   OOXXXXXOOO
                     OXXXXXXXOO
                      XXXXXXXXXOO
                      OOXXXXXXXXXOOO
                        OOXXXXXXXXXXOO
                          OXXXXXXXXXXXOO
                           OOOXXXXXXXXXXOOO
                              OOXXXXXXXXXXXOO
                                OOXXXXXXXXXXXOOO
                                  OOXXXXXXXXXXXXOO
                                    OOXXXXXXXXXXXXOOOOOOOOO
                                      OOXXXXXXXXXXXXXXXXXXXOOO
                                        OOXXXXXXXXXXXXXXXXXXXXOO
                                          OOOXXXXXXXXXXXXXXXXXXXOOO
                                             OOOOOXXXXXXXXXXXXXXXXXXOOO
                                                  XXXXXXXXXXXXXXXXXXXXOO
                                                  OOOOOXXXXXXXXXXXXXXXXXXOOO
                                                       OOOOXXXXXXXXXXXXXXXXXOOO
                                                           OOOOOXXXXXXXXXXXXXXXOO
                                                                OOOXXXXXXXXXXXXXOOO
                                                                   OOOOOXXXXXXXXXXXOOO
                                                                        OOOOOOOOXXXXXXOO
                                                                                OOOOOOOOOOOOOOOOOOOOO
```

Execution complete, time = 4.39 seconds
---

by: KW 93-1033
1/25/94



Deep circles still low FS.
Try 2:1.


by: KWI 93-1055
1/25/94
2:1 SLOPE
φ = 35° 0 COHESION
γ = 125 pcf
50,5
80,20
84,20
114,35
1.84
1.74
1.60
1.43*
1.41*
* Results attached

by: KW 93-1033
1/25/94
Checked 1-27-94

1033RIVR.DAT

SANDY ZONE CASE -- DYSART DRAIN EVALUATION AT 2:1 -- BENCHED
2 0 1 6 2 0 0 0 0 0 0
3 0 0 0 0 0 2116 62.4 0 0
0 0 10 0 0
130 -25 1
35
0 10 10 5 150
50 10 10 5 150
80 20 20 20 150
84 20 20 20 150
114 35 35 35 150
175 35 35 35 150
1 125 0.01 35 0
STOP

by: KW 93-1033
1/25/94
1 OF 4
Checked [illegible]
1-27-94

```
*************************************************
* ********************************************* *
* *                                           * *
* *    --------------------------------       * *
* *     TSTAB  slope stability analysis       * *
* *        Revision 2.40 - 02/12/85           * *
* *    --------------------------------       * *
* *                                           * *
* *    TAGA Engineering Software Services     * *
* *           Berkeley, California            * *
* *                                           * *
* *   IBM PC & 8086/8088 MS-DOS Version by    * *
* *                                           * *
* * Design Professionals Management Systems   * *
* *           Kirkland, Washington            * *
* *                                           * *
* *      copyright (c) 1983, 1984 TAGA        * *
* *      copyright (c) 1983, 1984 DPMS        * *
* *                                           * *
* ********************************************* *
*************************************************
```

```
*************************************************************************
  SANDY ZONE CASE -- DYSART DRAIN EVALUATION AT 2:1 -- BENCHED
*************************************************************************
```

```
*********************************
  ANALYSIS BY SPENCER'S METHOD
*********************************
```

```
**************
  INPUT DATA
**************
```

```
 CONTROL DATA,
      AUTOMATIC SEARCH FOR CRITICAL CIRCLE
      NUMBER OF DEPTH LIMITING TANGENTS                 1
      NUMBER OF VERTICAL SECTIONS                       6
      NUMBER OF SOIL LAYER BOUNDARIES                   2
      NUMBER OF POINTS DEFINING COHESION PROFILE        0
      NUMBER OF CURVES DEFINING COHESION ANISOTROPY     0
      NUMBER OF BOUNDARY LINE LOADS                     0
      NUMBER OF BOUNDARY PRESSURE LOADS                 0

      SEISMIC COEFFICIENT                    =       .000
      ATMOSPHERIC PRESSURE                   =   2116.000
      UNIT WEIGHT OF WATER                   =     62.400
      UNIT WEIGHT OF WATER IN TENSION CRACK  =     62.400
      INITIAL ESTIMATE OF F                  =      3.000
      INITIAL ESTIMATE OF THETA              =     10.000
      ALLOWABLE FORCE IMBALANCE              =     10.000
      ALLOWABLE MOMENT IMBALANCE             =    100.000

 SEARCH STARTS AT CENTER ( 130.0, -25.0),WITH FINAL GRID OF  1.0

 ALL CIRCLES TANGENT TO DEPTH,  35.0,

 GEOMETRY

     SECTIONS        .00    50.00    80.00    84.00   114.00   175.00

     T. CRACKS     10.00    10.00    20.00    20.00    35.00    35.00
     W IN CRACK    10.00    10.00    20.00    20.00    35.00    35.00
     BOUNDARY 1     5.00     5.00    20.00    20.00    35.00    35.00
```

by: KKl 93-1033
1/25/94
2 OF 4
Checked 1-27-94

BOUNDARY 2 150.00 150.00 150.00 150.00 150.00 150.00

SOIL PROPERTIES

LAYER	DENSITY	COHESION	FRICTION ANGLE	DELTA PHI
1	125.00	.01	35.00	.00

1

****
RESULTS
****

****
DEPTH LIMITING TANGENT NO. 1 AT Y = 35.00
****

1	35.0	60.0	130.0	-25.0	CIRCLE OUTSIDE SLOPE
2	35.0	60.0	128.0	-25.0	1.406
3	35.0	62.0	130.0	-27.0	CIRCLE OUTSIDE SLOPE
4	35.0	60.0	132.0	-25.0	CIRCLE OUTSIDE SLOPE
5	35.0	58.0	130.0	-23.0	CIRCLE OUTSIDE SLOPE
6	35.0	60.0	126.0	-25.0	1.423
7	35.0	62.0	128.0	-27.0	1.408
8	35.0	58.0	128.0	-23.0	CIRCLE OUTSIDE SLOPE
9	35.0	60.0	127.0	-25.0	1.413
10	35.0	61.0	128.0	-26.0	1.406
11	35.0	60.0	129.0	-25.0	CIRCLE OUTSIDE SLOPE
12	35.0	59.0	128.0	-24.0	CIRCLE OUTSIDE SLOPE
13	35.0	61.0	127.0	-26.0	1.415
14	35.0	61.0	129.0	-26.0	CIRCLE OUTSIDE SLOPE
15	35.0	59.0	129.0	-24.0	CIRCLE OUTSIDE SLOPE
16	35.0	59.0	127.0	-24.0	1.411

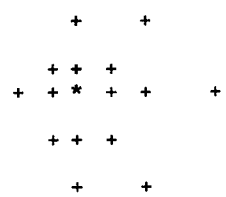
F.S. MINIMUM= 1.406 FOR THE CIRCLE OF CENTER (128.0, -25.0)

****
DETAIL OF CALCULATION FOR CRITICAL CIRCLE
****

ITERATION	F	THETA DEGREES	EXCESS FORCE	EXCESS MOMENT
1	3.000	10.0	-8.3	89.4
2	2.000	10.0	-4.5	48.3
3	1.520	20.0	-1.1	-9.1
4	1.405	18.9	.0	-.1
5	1.406	26.5	.0	.0
6	1.406	26.5	.0	.0

1

****
LOCATION OF CRITICAL CIRCLE
****



by: KW 93-1033
1/25/94
3 OF 4
Checked By
1-27-94

By. KW 93-1033
1/25/94

4 OF 4

Checked KEA 1-27-94

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-----------------------------------------
Execution complete, time =     1.81 seconds
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```
SANDY ZONE CASE -- DYSART DRAIN EVALUATION AT 2:1 -- BENCHED
2 0 1 6 2 0 0 0 0 0 0 0
3 0 0 0 0 0 2116 62.4 0 0
0 0 10 0 0
130 -25 1
35
0    10 10 5  150
50   10 10 5  150
80   20 20 20 150
84   20 20 20 150
114  35 35 35 150
175  35 35 35 150
1 125 0.01 35 0
STOP
```

by: KW 93-1033
1/25/94

1033RIV1.DAT

Check [illegible] 1-27-94

SANDY ZONE CASE -- DYSART DRAIN EVALUATION AT 2:1 -- BENCHED
2 0 0 6 2 0 0 0 0 0 0 0
3 0 0 0 0 0 2116 62.4 0 0
0 0 10 0 0
135 -40 1
95 27.5
0 10 10 5 150
50 10 10 5 150
80 20 20 20 150
84 20 20 20 150
114 35 35 35 150
175 35 35 35 150
1 125 0.01 35 0
STOP

by: KW 93-1033
1/25/94
1 OF 4
Check [illegible] 1-27-94

```
*************************************************
* ********************************************* *
* *                                           * *
* *    ---------------------------------      * *
* *     TSTAB  slope stability analysis       * *
* *        Revision 2.40 - 02/12/85           * *
* *    ---------------------------------      * *
* *                                           * *
* *    TAGA Engineering Software Services     * *
* *           Berkeley, California            * *
* *                                           * *
* *   IBM PC & 8086/8088 MS-DOS Version by    * *
* *                                           * *
* * Design Professionals Management Systems   * *
* *           Kirkland, Washington            * *
* *                                           * *
* *      copyright (c) 1983, 1984 TAGA        * *
* *      copyright (c) 1983, 1984 DPMS        * *
* *                                           * *
* ********************************************* *
*************************************************
```

```
*************************************************************************
  SANDY ZONE CASE -- DYSART DRAIN EVALUATION AT 2:1 -- BENCHED
*************************************************************************
```

```
*********************************
  ANALYSIS BY SPENCER'S METHOD
*********************************
```

```
**************
  INPUT DATA
**************
```

```
 CONTROL DATA,
      AUTOMATIC SEARCH FOR CRITICAL CIRCLE
      NUMBER OF DEPTH LIMITING TANGENTS                 0
      NUMBER OF VERTICAL SECTIONS                       6
      NUMBER OF SOIL LAYER BOUNDARIES                   2
      NUMBER OF POINTS DEFINING COHESION PROFILE        0
      NUMBER OF CURVES DEFINING COHESION ANISOTROPY     0
      NUMBER OF BOUNDARY LINE LOADS                     0
      NUMBER OF BOUNDARY PRESSURE LOADS                 0

      SEISMIC COEFFICIENT                    =       .000
      ATMOSPHERIC PRESSURE                   =   2116.000
      UNIT WEIGHT OF WATER                   =     62.400
      UNIT WEIGHT OF WATER IN TENSION CRACK  =     62.400
      INITIAL ESTIMATE OF F                  =      3.000
      INITIAL ESTIMATE OF THETA              =     10.000
      ALLOWABLE FORCE IMBALANCE              =     10.000
      ALLOWABLE MOMENT IMBALANCE             =    100.000

 SEARCH STARTS AT CENTER ( 135.0, -40.0),WITH FINAL GRID OF  1.0

 ALL CIRCLES PASS THROUGH THE POINT (  95.0,  27.5)

 GEOMETRY

     SECTIONS        .00    50.00    80.00    84.00   114.00   175.00

     T. CRACKS     10.00    10.00    20.00    20.00    35.00    35.00
     W IN CRACK    10.00    10.00    20.00    20.00    35.00    35.00
     BOUNDARY 1     5.00     5.00    20.00    20.00    35.00    35.00
```

by. KW 93-1037
1/25/94
2 of 4
Check KLR 1-27-94

BOUNDARY 2 150.00 150.00 150.00 150.00 150.00 150.00

SOIL PROPERTIES

LAYER	DENSITY	COHESION	FRICTION ANGLE	DELTA PHI
1	125.00	.01	35.00	.00

1

****
RESULTS
****

NUMBER	TANGENT	RADIUS	(X) CENTER	(Y) CENTER	F.S.
1	38.5	78.5	135.0	-40.0	5.395
2	37.5	77.5	133.0	-40.0	4.028
3	38.2	80.2	135.0	-42.0	5.084
4	39.5	79.5	137.0	-40.0	6.854
5	38.7	76.7	135.0	-38.0	5.699
6	36.5	76.5	131.0	-40.0	1.437
7	37.2	79.2	133.0	-42.0	1.435
8	37.7	75.7	133.0	-38.0	4.329
9	36.3	78.3	131.0	-42.0	1.440
10	37.0	81.0	133.0	-44.0	1.436
11	38.2	80.2	135.0	-42.0	5.084
12	36.7	78.7	132.0	-42.0	1.436
13	37.1	80.1	133.0	-43.0	1.435
14	37.7	79.7	134.0	-42.0	4.390
15	37.3	78.3	133.0	-41.0	1.434
16	36.9	77.9	132.0	-41.0	1.435
17	37.8	78.8	134.0	-41.0	4.544
18	37.5	77.5	133.0	-40.0	4.028
19	36.7	78.7	132.0	-42.0	1.436
20	37.7	79.7	134.0	-42.0	4.390
21	38.0	78.0	134.0	-40.0	4.698
22	37.0	77.0	132.0	-40.0	1.435

F.S. MINIMUM= 1.434 FOR THE CIRCLE OF CENTER (133.0, -41.0)

****
DETAIL OF CALCULATION FOR CRITICAL CIRCLE
****

ITERATION	F	THETA DEGREES	EXCESS FORCE	EXCESS MOMENT
1	3.000	10.0	-1186.7	12594.8
2	2.000	9.8	-631.2	6365.6
3	1.535	19.8	-144.3	-1415.0
4	1.442	29.8	-6.5	104.2
5	1.434	26.2	.0	.4

1
****
LOCATION OF CRITICAL CIRCLE
****

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+ ++ + +  +

  +   +
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by. KW 93-1033
1/25/94
3 of 4
Check KZA 1-27-94

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by: KW 93-1033
1/25/94
4 of 4
Check KW 1-27-94

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-------------------------------------------
Execution complete, time =     3.13 seconds
-------------------------------------------
```

by: KW 93-1033

1/24/94

Checked 1-27-94

Pavement Design

Traffic Projection

from MAG Transportation & Planning Office

Average Weekday Traffic Maps

Year	Dysart	El Mirage	Reems	Northern
1990	5,000	2,000	1,000	2,000
2010	18,000	8,000	1,000	3,000
1994*	7,600	3,200	1,000	2,200
2014**	20,600	9,200	1,000	3,200

* by interpolation

** by extrapolation

Assume 1 lane each direction (current configuration), except Dysart

Assume truck percentages as follows

Road	Trucks (Above LT)
Dysart	5%
El Mirage	5%
Reems	7%
Northern	7%

ESAL Calculations following pages.

Computation of Equivalent Single Axle Loads

Computation for:	Dysart Road	Project No.: 93-1033	By: KW	Date:	25-Jan-94

Initial Design Year:	1994
Design Period (yrs.):	20
Total Initial ADT:	7600
Total Final ADT:	20600
Pavement Type:	Flexible
ADT is 1-Way or 2-Way:	2
Lanes -- one way:	2
Design Lane Factor:	0.45
Average Design ADT:	6345

Dysart Drain crossing

	Vehicle Classification							
	Auto	Trucks						Total
Percent Traffic:	95.00%	5.00%						
Vehicle Equivalence:	0.00080	0.45000						
ESAL for Design Period:	35, 202	1, 042, 166						1, 077, 368

Computation of Equivalent Single Axle Loads

Computation for:	El Mirage Road	Project No.: 93-1033	By: KW	Date: 25-Jan-94

Initial Design Year:	1994	Dysart Drain crossing
Design Period (yrs.):	20	
Total Initial ADT:	3200	
Total Final ADT:	9200	
Pavement Type:	Flexible	
ADT is 1-Way or 2-Way:	2	
Lanes -- one way:	1	
Design Lane Factor:	0.5	
Average Design ADT:	3100	

Vehicle Classification

	Auto	Trucks						Total
Percent Traffic:	95.00%	5.00%						
Vehicle Equivalence:	0.00080	0.45000						
ESAL for Design Period:	17, 199	509, 175						526, 374

Computation of Equivalent Single Axle Loads

Computation for:	Northern Avenue	Project No.: 93-1033	By: KW	Date: 25-Jan-94

Initial Design Year:	1994	near detention basin
Design Period (yrs.):	20	
Total Initial ADT:	2200	
Total Final ADT:	3200	
Pavement Type:	Flexible	
ADT is 1-Way or 2-Way:	2	
Lanes -- one way:	1	
Design Lane Factor:	0.5	
Average Design ADT:	1350	

Vehicle Classification

	Auto	Trucks						Total
Percent Traffic:	93.00%	7.00%						
Vehicle Equivalence:	0.00080	0.45000						
ESAL for Design Period:	7, 332	310, 432						317, 765

Computation of Equivalent Single Axle Loads

Computation for:	Reems Road	Project No.: 93-1033	By: KW	Date: 25-Jan-94

Initial Design Year:	1994	near detention basin
Design Period (yrs.):	20	
Total Initial ADT:	1000	
Total Final ADT:	1000	
Pavement Type:	Flexible	
ADT is 1-Way or 2-Way:	2	
Lanes -- one way:	1	
Design Lane Factor:	0.5	
Average Design ADT:	500	

	Vehicle Classification							
	Auto	Trucks						Total
Percent Traffic:	90.00%	10.00%						
Vehicle Equivalence:	0.00080	0.45000						
ESAL for Design Period:	2, 628	164, 250						166, 878

by: KW 93-1033

1/25/94

Chuck [illegible] 1-27-94

Pavement Design Factors

Road	ADT Range	Class	Z_r	P_o	P_t	ΔPSI
Dysart	10,000 +	2	-1.645	4.2	2.8	1.4
El Mirage	2001-10,000	3	-1.282	4.1	2.6	1.5
Northern	2,001-10,000	3	-1.282	4.1	2.6	1.5
Reems	501-2,000	4	-1.037	4.1	2.6	1.5

Asphalt Concrete Mat'l coefficient = 0.44 } Assumed
ABC " " = 0.14 }

Assume drainage is fair. ($m_2 = 1.0$)

For project in Glendale

SVF = 1

Index of Retained Strength = 40%

Use $S_o = 0.45$ (AASHTO policy)

From ADOT Table 202.04-1

Road	Min. SN	Min AC Thick	Remarks
Dysart	3.5	5.0"	Urban 10,000+ ADT M
El Mirage	2.75	4.0"	Urban Main 2001-1000
Northern	2.5	4.0"	Rural Main "
Reems	2.0	3.0"	Rural Main 500-200

by: KW 93-1033

1/25/94

Check 1-27-94

R-Values

Dysart:

D 0-5' R = 27 TESTED

E 0-5' %-#200 = 41, PI = 11 R = 37 CORRELATED (ADOT)

USE 27

Reems/Northern

A, C tested, 33 = minimum R.

USE 33

El Mirage

G 0-5' R = 64 TESTED (granular)

F 0-5' %-#200 = 41 PI = 19 R = 27 CORRELATED (ADOT)

USE 27

Flexible Pavement Design

DESIGN FOR:	*Dysart Road*	
PROJECT NO.:	93-1033	25-Jan-94
1. Traffic & Highway Data-		
18-kip Equivalent Single Axle Load Applications:	1, 077, 368	Design
THA Highway Class (1, 2, 3, 4, 5):	2	Comments
2. Project Design Criteria-		
Initial Design Serviceability - Po:	4.2	
Design Serviceability - Pt:	2.8	
Pavement Material Coefficients:		
a. Asphalt Concrete (0.44 max.):	0.44	
b. CTB or BTB (0.28 max.):	0	
c. Cement/Lime treated subgrade (0.23 max):	0	
d. Aggregate Base (0.14 max.):	0.14	
e. Aggregate Subbase (0.11 max.):	0	
3. Drainage Characteristics:		
Excellent(1); Good (2); Fair (3); Poor (4); Very Poor (5)		
a. Base Course:	3	
b. Subbase Course - Layer 1:	0	
c. Subbase Course - Layer 2:	0	
4. Location:	Glendale	
5. Calculation Section:		
SVF =	1	
Zr =	-1.645	
Index Ret. Strength =	40	
So =	0.45	
Quality of Drainage		
Base Course (m2)	1	
Subbase Course-1 (m3)	Not Used	
Subbase Course-2 (m3)	Not Used	
R-mean =	27	
Resilient Modulus =	16066	
Enter Initial SN Guess and modify until RHS = LHS		
Guess at SN =	2.86	
RHS =	6.032	
LHS =	6.032	
Minimum SN =	3.5	
Minimum Pavement Thickness =	5	
Actual SN =	3.500	Use minimum

Flexible Pavement Design Table	
Dysart Road	
Thickness of Asphalt Concrete (inches)	Thickness of Base Course (inches)
8	0
7.5	4
7	4
6.5	5
6	7
5.5	8
5	10
5	10
5	10
5	10
5	10
5	10
5	10
5	10
5	10
5	10
5	10
5	10
5	10
5	10
5	10

Flexible Pavement Design

DESIGN FOR:	*El Mirage Road*	
PROJECT NO.:	93-1033	25-Jan-94
1. Traffic & Highway Data-		
18-kip Equivalent Single Axle Load Applications:	526, 374	Design
THA Highway Class (1, 2, 3, 4, 5):	3	Comments
2. Project Design Criteria-		
Initial Design Serviceability - Po:	4.1	
Design Serviceability - Pt:	2.6	
Pavement Material Coefficients:		
a. Asphalt Concrete (0.44 max.):	0.44	
b. CTB or BTB (0.28 max.):	0	
c. Cement/Lime treated subgrade (0.23 max):	0	
d. Aggregate Base (0.14 max.):	0.14	
e. Aggregate Subbase (0.11 max.):	0	
3. Drainage Characteristics:		
Excellent(1); Good (2); Fair (3); Poor (4); Very Poor (5)		
a. Base Course:	3	
b. Subbase Course - Layer 1:	0	
c. Subbase Course - Layer 2:	0	
4. Location:	Glendale	
5. Calculation Section:		
SVF =	1	
Zr =	-1.282	
Index Ret. Strength =	40	
So =	0.45	
Quality of Drainage		
Base Course (m2)	1	
Subbase Course-1 (m3)	Not Used	
Subbase Course-2 (m3)	Not Used	
R-mean =	27	
Resilient Modulus =	16066	
Enter Initial SN Guess and modify until RHS = LHS		
Guess at SN =	2.35	
RHS =	5.721	
LHS =	5.721	
Minimum SN =	2.75	
Minimum Pavement Thickness =	4	
Actual SN =	2.750	Use minimum

Flexible Pavement Design Table	
El Mirage Road	
Thickness of Asphalt Concrete (inches)	Thickness of Base Course (inches)
6.5	0
6	4
5.5	4
5	4
4.5	6
4	8*
4	8
4	8
4	8
4	8
4	8
4	8
4	8
4	8
4	8
4	8
4	8
4	8
4	8
4	8

* 10 in is MCDOT minimum.

Flexible Pavement Design

DESIGN FOR:	*Northern Avenue*	
PROJECT NO.:	93-1033	25-Jan-94
1. Traffic & Highway Data-		
18-kip Equivalent Single Axle Load Applications:	317, 765	Design
THA Highway Class (1, 2, 3, 4, 5):	3	Comments
2. Project Design Criteria-		
Initial Design Serviceability - Po:	4.1	
Design Serviceability - Pt:	2.6	
Pavement Material Coefficients:		
a. Asphalt Concrete (0.44 max.):	0.44	
b. CTB or BTB (0.28 max.):	0	
c. Cement/Lime treated subgrade (0.23 max):	0	
d. Aggregate Base (0.14 max.):	0.14	
e. Aggregate Subbase (0.11 max.):	0	
3. Drainage Characteristics:		
Excellent(1); Good (2); Fair (3); Poor (4); Very Poor (5)		
a. Base Course:	3	
b. Subbase Course - Layer 1:	0	
c. Subbase Course - Layer 2:	0	
4. Location:	Glendale	
5. Calculation Section:		
SVF =	1	
Zr =	-1.282	
Index Ret. Strength =	40	
So =	0.45	
Quality of Drainage		
Base Course (m2)	1	
Subbase Course-1 (m3)	Not Used	
Subbase Course-2 (m3)	Not Used	
R-mean =	33	
Resilient Modulus =	19756	
Enter Initial SN Guess and modify until RHS = LHS		
Guess at SN =	1.984	
RHS =	5.502	
LHS =	5.502	
Minimum SN =	2.5	
Minimum Pavement Thickness =	4	
Actual SN =	2.500	Use minimum

Flexible Pavement Design Table	
Northern Avenue	
Thickness of Asphalt Concrete (inches)	Thickness of Base Course (inches)
6	0
5.5	4
5	4
4.5	4
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6
4	6

MCDOT minimum = 3" on 10"

$SN = 3(0.44) + 10(0.14) = 2.72 > 1.98$ OK

Flexible Pavement Design

DESIGN FOR:	*Reems Road*	
PROJECT NO.:	93-1033	25-Jan-94
1. Traffic & Highway Data-		
18-kip Equivalent Single Axle Load Applications:	166, 878	Design
THA Highway Class (1, 2, 3, 4, 5):	4	Comments
2. Project Design Criteria-		
Initial Design Serviceability - Po:	4.1	
Design Serviceability - Pt:	2.6	
Pavement Material Coefficients:		
a. Asphalt Concrete (0.44 max.):	0.44	
b. CTB or BTB (0.28 max.):	0	
c. Cement/Lime treated subgrade (0.23 max):	0	
d. Aggregate Base (0.14 max.):	0.14	
e. Aggregate Subbase (0.11 max.):	0	
3. Drainage Characteristics:		
Excellent(1); Good (2); Fair (3); Poor (4); Very Poor (5)		
a. Base Course:	3	
b. Subbase Course - Layer 1:	0	
c. Subbase Course - Layer 2:	0	
4. Location:	Glendale	
5. Calculation Section:		
SVF =	1	
Zr =	-1.037	
Index Ret. Strength =	40	
So =	0.45	
Quality of Drainage		
Base Course (m2)	1	
Subbase Course-1 (m3)	Not Used	
Subbase Course-2 (m3)	Not Used	
R-mean =	33	
Resilient Modulus =	19756	
Enter Initial SN Guess and modify until RHS = LHS		
Guess at SN =	1.695	
RHS =	5.222	
LHS =	5.222	
Minimum SN =	2	
Minimum Pavement Thickness =	3	
Actual SN =	2.000	Use minimum

Flexible Pavement Design Table	
Reems Road	
Thickness of Asphalt Concrete (inches)	Thickness of Base Course (inches)
4.5	0
4	4
3.5	4
3	5
3	5
3	5
3	5
3	5
3	5
3	5
3	5
3	5
3	5
3	5
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3	5
3	5
3	5

MCDOT Minimum = 3" on 10"

SN = 2.72 > 1.70 OK

by: KW 93-1033

12/13/93

Check 1-26-94

Shallow Foundation - Walls : Wingwalls

Clays:

$\phi = 30°$ $c = 200psf$ $\gamma = 120pcf$ $N_c = 32$ $N_q = 20$ $N_\gamma = 17$ (DM7.2-13

$q_{ULT} = cN_c(1+.3\frac{B}{L}) + \gamma D N_q + 0.4 \gamma B N_\gamma$

For 1.33' width; "long" footing; ignore correction factor

1.5' $q_{ULT} = (1.0)(200)(32) + (120)(1.5)(20) + (0.4)(120)(1.33)(17) = 11085$ psf

2' $q_{ULT} = (1.0)(200)(32) + (120)(2)(20) + (0.4)(120)(1.33)(17) = 12285$ psf

3' $q_{ULT} =$ " $+ (120)(3)(20) +$ " $= 14685$ psf

4' $q_{ULT} =$ " $+ (120)(4)(20) +$ " $= 17085$ psf

DEPTH	FS = 3 $\frac{1}{3} q_{ULT}$	Reduce for settlement
1.5'	3700 psf	2000 psf
2'	4100 psf	2250 psf
3'	4900 psf	2500 psf
4'	5700 psf	2750 psf

Sands:

$\phi = 35°$ $c = 0$ $\gamma = 125pcf$ $N_q = 35$ $N_\gamma = 40$

DEPTH	$\frac{1}{3} q_{ULT}$
1.5'	3100 psf
2'	3800 psf
3'	5260 psf
4'	6720 psf

REDULE AS ABOVE.

by: KW 93-1033

1/26/94

Check KSR 1-26-94

BRIDGES- SPREAD FOOTINGS

Supported at indicated depth below channel bottom.

Ignore $N\gamma$ term; then ignore wt. of foundation concrete below grade.

Clay soils

FS = 3

Depth	q_A from bearing capacity theory
4'	$q_A = \frac{1}{3}\left[(1.0)(200)(32) + (120)(4)(20)\right] = 5333$ psf
6'	$q_A = \frac{1}{3}\left[\quad " \quad + (120)(6)(20)\right] = 6933$ psf
8'	$q_A = \frac{1}{3}\left[\quad " \quad + (120)(8)(20)\right] = 8533$ psf

Check settlement: following pages

Allow > 4' width :

4'	4500 psf
6'	6000
8'	8000

For 2-4' width reduce by 500 for stability

For column footings increase by 500.

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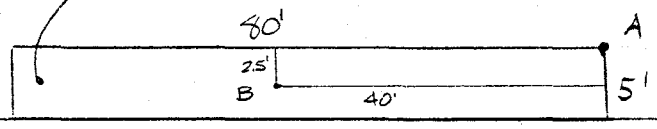
1/26/94
[illegible] 1-26-94

Assume σ_{APP} = 4500psf at 4'



A: $m = \frac{5}{z}$ $n = \frac{80}{z}$ $\Delta\sigma = I_\sigma \sigma_{APP}$

B: $m = \frac{2.5}{z}$ $n = \frac{40}{z}$ $\Delta\sigma = 4 I_\sigma \sigma_{APP}$

B:

	z BELOW FOOTING	m	n	Boussinesq I_σ	$\Delta\sigma$	$120(z+4)$ σ_{OVB}	σ_{FINAL}	(NEXT PAGE) ε	Thick	Incr. sett.
0										
	2'	1.25	20	.22	3960	720	4680	2.2%	48"	1.06"
4										
	6'	0.42	6.7	.12	2160	1200	3360	1.2%	48"	0.58"
8										
	10'	0.25	4	.075	1350	1680	3030	0.7%	48"	0.34"
12										
	14'	0.18	2.9	.057	1030	2160	3190	0.5%	48"	0.24"
16										
	18'	0.14	2.2	.042	760	2640	3400	0.3%	48"	0.14"
20										
	25'	0.10	1.6	.03	540	3480	4020	0.25%	120"	0.30"
30										
	35'	0.07	1.1	.024	430 ↙210%	4680	5110	0.2%	120"	0.24"
40										
										2.9"

A:

0										
	2'	2.5	40	0.244	1100	720	1820	0.8%	48"	0.38"
4										
	6'	0.83	13.3	0.187	840	1200	2040	0.5%	48"	0.24"
8										
	10	0.5	8	0.139	630	1680	2310	0.4%	48"	0.19"
12										
	14	0.36	5.7	0.105	470	2160	2630	0.2%	48"	0.10"
16										
	18	0.28	4.4	0.087	390	2640	3030	0.2%	48"	0.10"
20										
	25	0.20	3.2	0.062	280 ↙410%	3480	3760	0.1%	120"	0.12"
30										
										1.1"

FOOTING IS RIGID; Approx settlement = $\frac{1}{2}(2.9 + 1.1) = 2.0"$

Local experience indicates this procedure provides estimate from 1 to 4 X actual.

Estimated settlement = 0.5 - 1.0" SAY 3/4"

REPORT ON LABORATORY TESTS

LWI

SAMPLE: Date 1/26/94

CHECK 1-26-94

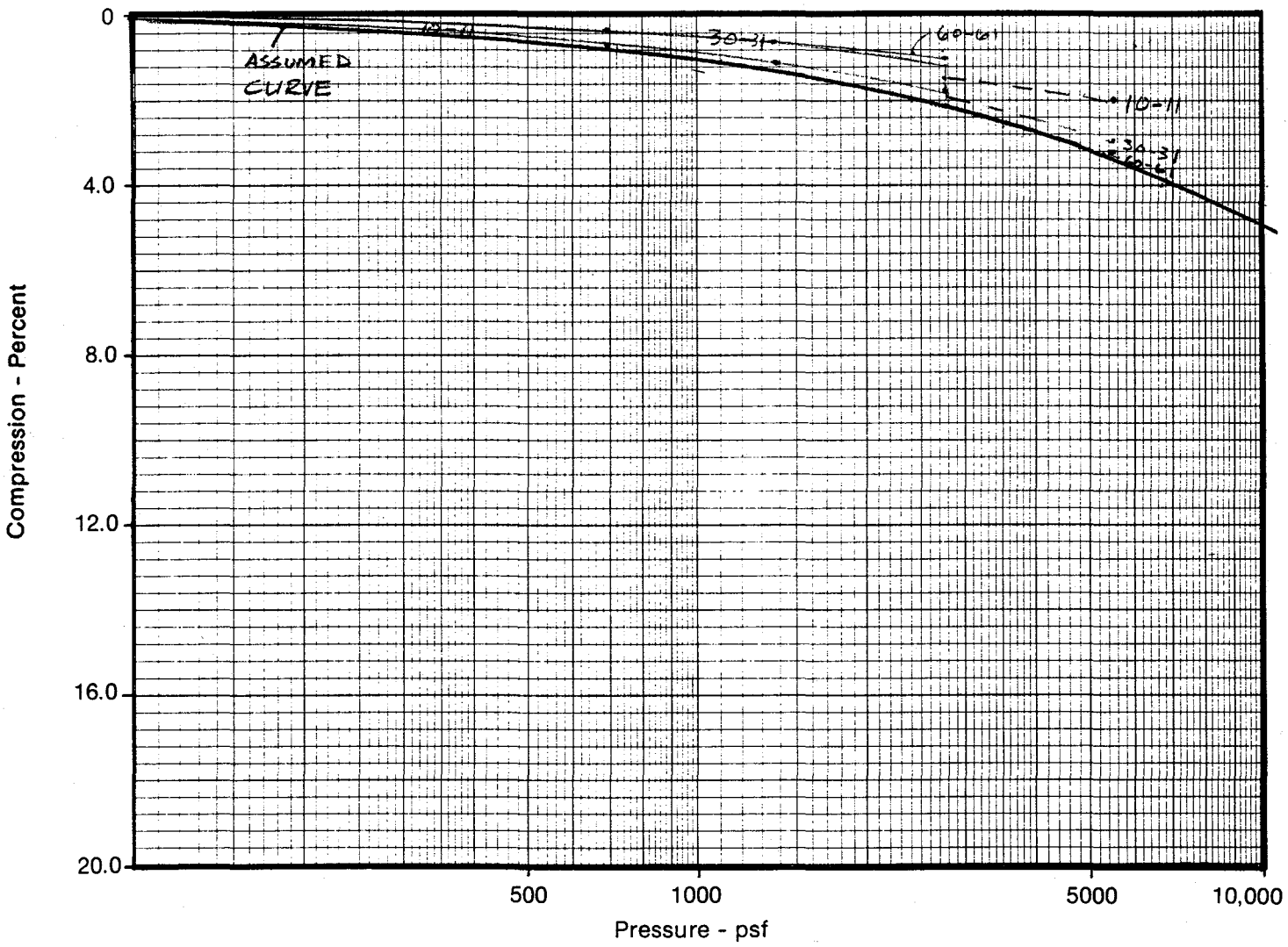
Source B 10 (DYSART ROAD)

Type Driven ring

Material Clay subgrade

Sampled By

TESTED:



Project No. 93-1033

THOMAS-HARTIG & ASSOCIATES, INC.

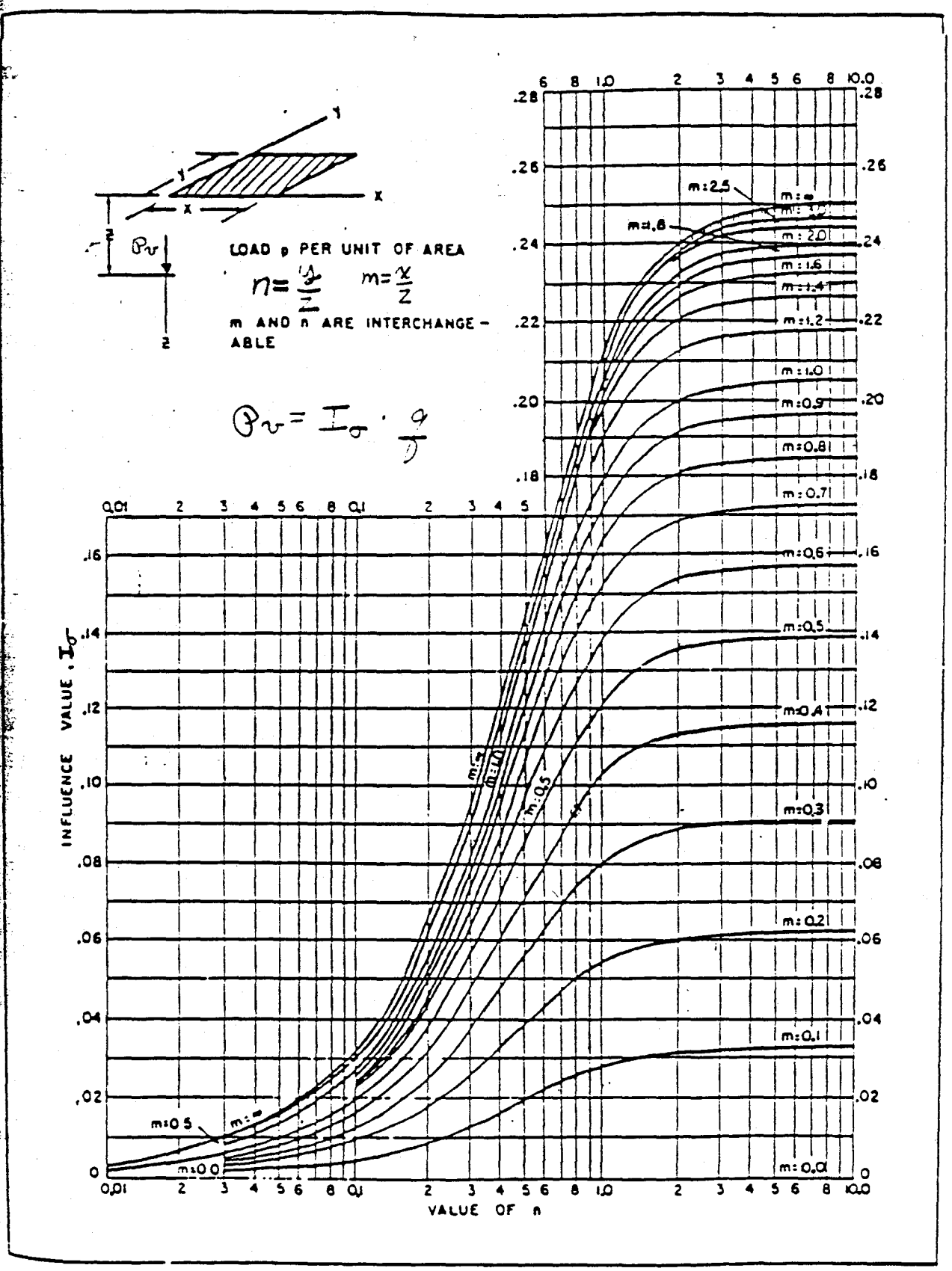


FIGURE 5-3

Influence Value for Vertical Stress Under Corner of Uniformly Loaded Rectangular Area

KW 93-1033
1/26/94
Check [illegible]

Elastic Check

$S = qB \frac{1-\mu^2}{E_s} I_w$ (Bowles)

for 4' depth $q = 4500$ psf

$B = 5$ ft

$I_w = 2.5$ for $\frac{L}{B} = 16$

$E_s = 4$ ksi $= 576$ ksf } Bowles, Sandy clay Tables 2.3 & 2.4

$\mu = 0.3$

$S = (4500 \frac{lb}{ft^2})(5 ft) \frac{1-0.3^2}{576000 \frac{lb}{ft^2}} (2.5)(12 \frac{in}{ft}) = 1.06''$ similar to consolidation analysis.

For sandy materials at El Mirage, gravelly in vicinity of flowline; expect $E_s = 3$ to 28 ksi; use recommendations for clay.

by: KW 93-1033

1/25/94

Check 1-26-94

DRILLED SHAFT CAPACITIES

Soil Characteristics

Clay soils: Use $\phi = 30°$ $c = 200$ psf

use $K = 0.7$ per DM-7, FHWA

DM-7.2 calls for $FS = 3$.

Local policy calls for $FS_{SKIN} = 2.5$ $FS_{TIP} = 3$ ← USE

$N_q = 10$

$N_c = 32$

$\gamma = 120$ pcf $= \gamma'$

MAXIMUM END BEARING = 8,500 psf

Granular soils: Use $\phi = 35°$ $c = 0$ $K = 0.7$

$N_q = 25$

$N_c = 0$

$\gamma = 120$ pcf $= \gamma'$

MAXIMUM END BEARING = 10,000 psf

El Mirage - 10 ft Clay over sands & gravels

Dysart - Clay soils

Location: El Mirage Road Bridge

Project No.: 93-1033

Computed By: KW

Date: 01/26/94

SUMMARY OF RESULTS

Pier Capacity Data:

Pier Diameter=	3	ft
FS Side Friction=	2.5	
FS End Bearing=	3	
Surf. Elevation=	1080	ft
Alpha=	0.45	

Settlement:

Ld Trnsfr =	0.65		
Axial Load =	350	kips	
Tip Load=	-301	kips	
Tip Pressure =	4.25	ksf	
Tip Modulus =	2000	ksf	
Pssn's Ratio =	0.2		
Iw =	0.88		
F3 =	0.5		
Settlement =	-0.32	inches or	-0.90% of diameter

NOT USED

Elevation	Depth	Soil Type	Allowable Increment Side Friction	Total Allowable Side Friction	Total Allowable End Bearing	Total Allowable Load
1080	0					
		CL	12.5	12.5	70.7	83.2
1070	10					
		SM	88.7	101.2	70.7	171.9
1050	30					
		SM	124.7	226.0	70.7	296.6
1035	45					
		SM	174.6	400.6	70.7	471.3
1020	60					

K	Phi	Su(ksf)	Unit Weight (pcf)	Eff. Stress	f(z)	Increment Length	Increment Diameter	Increment Side Friction	Total Ultimate Side Friction
0.7	30	0.2	120	0.60	0.332	10	3	31.34	31.34
				1.20					
0.7	35	0	120	2.40	1.176	20	3	221.74	253.07
				3.60					
0.7	35	0	120	4.50	2.206	15	3	311.82	564.89
				5.40					
0.7	35	0	120	6.30	3.088	15	3	436.54	1001.43
				7.20					

Total Allowable Side Friction	Base Area	Nc	Ngamma	Nq	q	qu	qa	Limiting End Bearing	Allowable End Bearing	Total Allowable Load
	7.07									
12.53	7.07	0	0	25	1.20	30.00	10.00	10	10.00	83.22
101.23	7.07	0	0	25	3.60	90.00	30.00	10	10.00	171.91
225.96	7.07	0	0	25	5.40	135.00	45.00	10	10.00	296.64
400.57	7.07	0	0	25	7.20	180.00	60.00	10	10.00	471.26

Location: El Mirage Road Bridge

Project No.: 93-1033

Computed By: KW

Date: 01/26/94

SUMMARY OF RESULTS

Pier Capacity Data:

Pier Diameter=	4	ft
FS Side Friction=	2.5	
FS End Bearing=	3	
Surf. Elevation=	1080	ft
Alpha=	0.45	

Settlement:

Id Trnsfr =	0.65	
Axial Load=	350	kips
Tip Load=	-518	kips
Tip Pressure =	-41.21	ksf
Tip Modulus =	2000	ksf
Pssn's Ratio =	0.2	
Iw =	0.88	
F3 =	0.5	

NOT USED

Settlement = -0.42 inches or -0.87% of diameter

Elevation	Depth	Soil Type	Allowable Increment Side Friction	Total Allowable Side Friction	Total Allowable End Bearing	Total Allowable Load
1080	0					
		CL	16.7	16.7	125.7	142.4
1070	10					
		SM	118.3	135.0	125.7	260.6
1050	30					
		SM	166.3	301.3	125.7	426.9
1035	45					
		SM	232.8	534.1	125.7	659.8
1020	60					

K	Phi	Su(ksf)	Unit Weight (pcf)	Eff. Stress	f(z)	Increment Length	Increment Diameter	Increment Side Friction	Total Ultimate Side Friction
0.7	30	0.2	120	0.60	0.332	10	4	41.78	41.78
				1.20					
0.7	35	0	120	2.40	1.176	20	4	295.65	337.43
				3.60					
0.7	35	0	120	4.50	2.206	15	4	415.76	753.19
				5.40					
0.7	35	0	120	6.30	3.088	15	4	582.06	1335.24
				7.20					

Total Allowable Side Friction	Base Area	Nc	Ngamma	Nq	q	qu	qa	Limiting End Bearing	Allowable End Bearing	Total Allowable Load
	12.57									
16.71	12.57	0	0	25	1.20	30.00	10.00	10	10.00	142.38
134.97	12.57	0	0	25	3.60	90.00	30.00	10	10.00	260.64
301.27	12.57	0	0	25	5.40	135.00	45.00	10	10.00	426.94
534.10	12.57	0	0	25	7.20	180.00	60.00	10	10.00	659.76

Location: El Mirage Road Bridge

Project No.: 93-1033

Computed By: KW

Date: 01/26/94

SUMMARY OF RESULTS

Pier Capacity Data:

Pier Diameter=	5	ft
FS Side Friction=	2.5	
FS End Bearing=	3	
Surf. Elevation=	1080	ft
Alpha=	0.45	

Settlement:

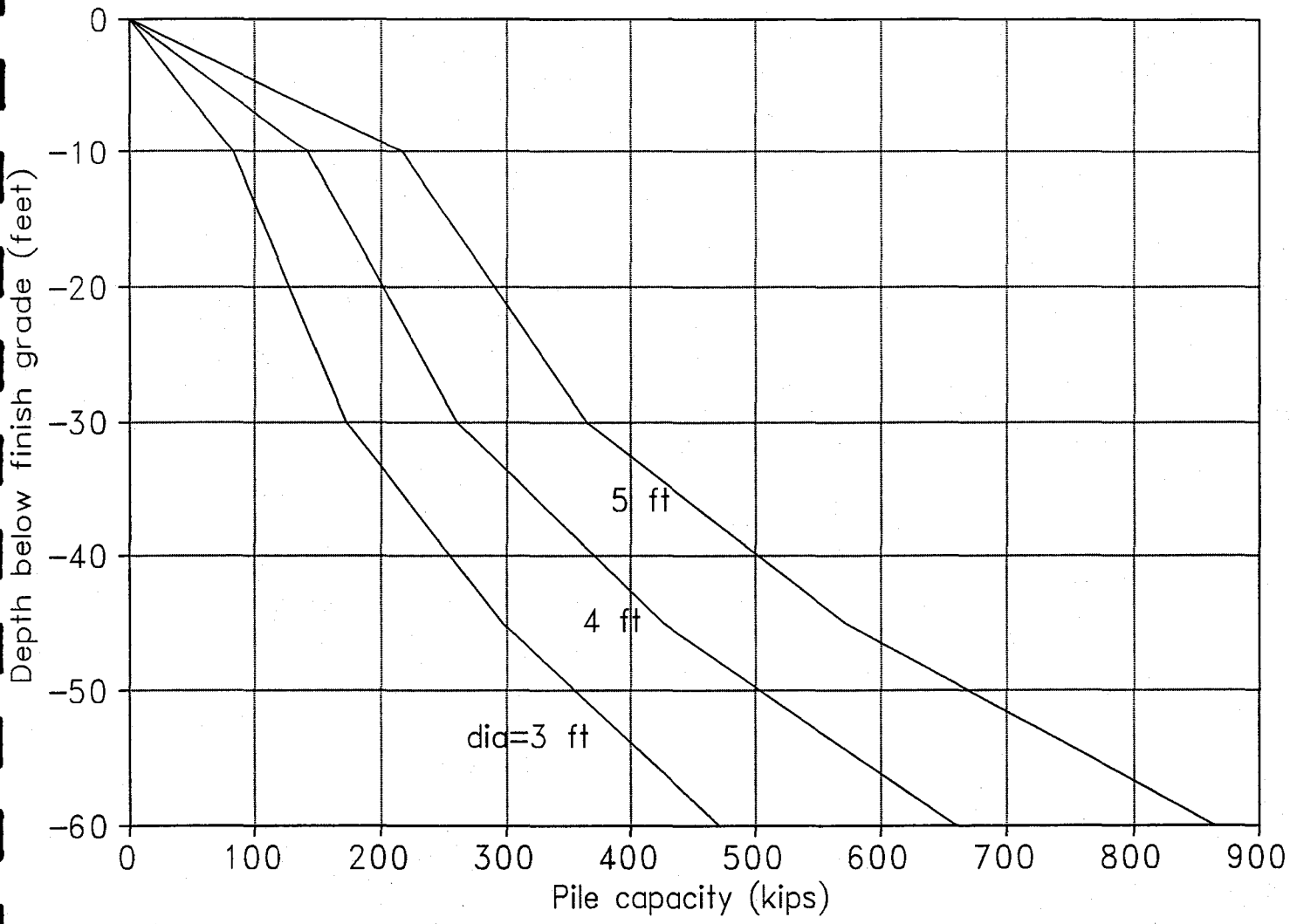
NOT USED

Ld Transfr =	0.65	
Axial Load =	350	kips
Tip Load=	35	kips
Tip Pressure =	-37.43	ksf
Tip Modulus =	2000	ksf
Pssn's Ratio =	0.2	
Iw =	0.88	
F3 =	0.5	
Settlement =	-0.47	inches or -0.79% of diameter

Elevation	Depth	Soil Type	Allowable Increment Side Friction	Total Allowable Side Friction	Total Allowable End Bearing	Total Allowable Load
1080	0					
		CL	20.9	20.9	196.3	217.2
1070	10					
		SM	147.8	168.7	196.3	365.1
1050	30					
		SM	207.9	376.6	196.3	572.9
1035	45					
		SM	291.0	667.6	196.3	864.0
1020	60					

K	Phi	Su(ksf)	Unit Weight (pcf)	Eff. Stress	f(z)	Increment Length	Increment Diameter	Increment Side Friction	Total Ultimate Side Friction
0.7	30	0.2	120	0.60	0.332	10	5	52.23	52.23
				1.20					
0.7	35	0	120	2.40	1.176	20	5	369.56	421.79
				3.60					
0.7	35	0	120	4.50	2.206	15	5	519.69	941.48
				5.40					
0.7	35	0	120	6.30	3.088	15	5	727.57	1669.06
				7.20					

Total Allowable Side Friction	Base Area	Nc	Ngamma	Nq	q	qu	qa	Limiting End Bearing	Allowable End Bearing	Total Allowable Load
	19.63									
20.89	19.63	0	0	25	1.20	30.00	10.00	10	10.00	217.24
168.72	19.63	0	0	25	3.60	90.00	30.00	10	10.00	365.06
376.59	19.63	0	0	25	5.40	135.00	45.00	10	10.00	572.94
667.62	19.63	0	0	25	7.20	180.00	60.00	10	10.00	863.97


0
-10
-20
-30
-40
-50
-60
Depth below finish grade (feet)
0
100
200
300
400
500
600
700
800
900
Pile capacity (kips)
5 ft
4 ft
dia=3 ft

Location: El Mirage Road Bridge

Project No.: 93-1033

Computed By: KW

Date: 01/26/94

SUMMARY OF RESULTS

Pier Capacity Data:

Pier Diameter=	4	ft
FS Side Friction=	2.5	
FS End Bearing=	3	
Surf. Elevation=	1080	ft
Alpha=	0.45	

CHECK SETTLEMENT

Settlement:

Ld Trnsfr =	0.6	from Reese & O'Neill (1989), Fig. 4 and s/d
Axial Load =	300	kips
Tip Load=	41	kips
Tip Pressure =	3.24	ksf
Tip Modulus =	432	ksf
Pssn's Ratio =	0.35	
Iw =	0.88	
F3 =	0.5	
Settlement =	0.14	inches or 0.29% of diameter

Elevation	Depth	Soil Type	Allowable Increment Side Friction	Total Allowable Side Friction	Total Allowable End Bearing	Total Allowable Load
1080	0					
		CL	16.7	16.7	125.7	142.4
1070	10					
		SM	27.9	44.7	125.7	170.3
1063	17					
		SM	49.7	94.3	125.7	220.0
1055	25					
		SM	78.5	172.8	125.7	298.5
1046	34					

K	Phi	Su(ksf)	Unit Weight (pcf)	Eff. Stress	f(z)	Increment Length	Increment Diameter	Increment Side Friction	Total Ultimate Side Friction
0.7	30	0.2	120	0.60	0.332	10	4	41.78	41.78
				1.20					
0.7	35	0	120	1.62	0.794	7	4	69.85	111.63
				2.04					
0.7	35	0	120	2.52	1.235	8	4	124.17	235.80
				3.00					
0.7	35	0	120	3.54	1.735	9	4	196.24	432.04
				4.08					

Total Allowable Side Friction	Base Area	Nc	Ngamma	Nq	q	qu	qa	Limiting End Bearing	Allowable End Bearing	Total Allowable Load
	12.57									
16.71	12.57	0	0	25	1.20	30.00	10.00	10	10.00	142.38
44.65	12.57	0	0	25	2.04	51.00	17.00	10	10.00	170.32
94.32	12.57	0	0	25	3.00	75.00	25.00	10	10.00	219.98
172.82	12.57	0	0	25	4.08	102.00	34.00	10	10.00	298.48

Location: Dysart Road Bridge

Project No.: 93-1033

Computed By: KW

Date: 01/26/94

SUMMARY OF RESULTS

Pier Capacity Data:

Pier Diameter=	3	ft
FS Side Friction=	2.5	
FS End Bearing=	3	
Surf. Elevation=	1080	ft
Alpha=	0.45	

Settlement:

Dl Trnsfr =	0.65	
Axial Load =	350	kips
Tip Load=	-218	kips
Tip Pressure =	-30.82	ksf
Tip Modulus =	2000	ksf
Pssn's Ratio =	0.2	
Iw =	0.88	
F3 =	0.5	
Settlement =	-0.23	inches or -0.65% of diameter

NOT USED

Elevation	Depth	Soil Type	Allowable Increment Side Friction	Total Allowable Side Friction	Total Allowable End Bearing	Total Allowable Load
1080	0					
		CL-CH	25.7	25.7	60.1	85.7
1065	15					
		CL-CH	66.8	92.5	60.1	152.5
1050	30					
		CL-CH	107.9	200.4	60.1	260.5
1035	45					
		CL-CH	149.1	349.5	60.1	409.5
1020	60					

K	Phi	Su(ksf)	Unit Weight (pcf)	Eff. Stress	f(z)	Increment Length	Increment Diameter	Increment Side Friction	Total Ultimate Side Friction
0.7	30	0.2	120	0.90	0.454	15	3	64.14	64.14
				1.80					
0.7	30	0.2	120	2.70	1.181	15	3	166.99	231.13
				3.60					
0.7	30	0.2	120	4.50	1.909	15	3	269.83	500.96
				5.40					
0.7	30	0.2	120	6.30	2.636	15	3	372.67	873.63
				7.20					

Total Allowable Side Friction	Base Area	Nc	Ngamma	Nq	q	qu	qa	Limiting End Bearing	Allowable End Bearing	Total Allowable Load
	7.07									
25.66	7.07	32	0	10	1.80	26.32	8.77	8.5	8.50	85.74
92.45	7.07	32	0	10	3.60	44.32	14.77	8.5	8.50	152.54
200.38	7.07	32	0	10	5.40	62.32	20.77	8.5	8.50	260.47
349.45	7.07	32	0	10	7.20	80.32	26.77	8.5	8.50	409.54

Location: Dysart Road Bridge

Project No.: 93-1033

Computed By: KW

Date: 01/26/94

SUMMARY OF RESULTS

Pier Capacity Data:

Pier Diameter=	4	ft
FS Side Friction=	2.5	
FS End Bearing=	3	
Surf. Elevation=	1080	ft
Alpha=	0.45	

Settlement:

Ld Trnsfr =	0.65	
Axial Load =	350	kips
Tip Load=	-40%	kips
Tip Pressure =	-32.40	ksf
Tip Modulus =	2000	ksf
Pssn's Ratio =	0.2	
Iw =	0.88	
F3 =	0.5	
Settlement =	-0.33	inches or -0.68% of diameter

NOT USED

Elevation	Depth	Soil Type	Allowable Increment Side Friction	Total Allowable Side Friction	Total Allowable End Bearing	Total Allowable Load
1080	0					
		CL-CH	34.2	34.2	106.8	141.0
1065	15					
		CL-CH	89.1	123.3	106.8	230.1
1050	30					
		CL-CH	143.9	267.2	106.8	374.0
1035	45					
		CL-CH	198.8	465.9	106.8	572.8
1020	60					

K	Phi	Su(ksf)	Unit Weight (pcf)	Eff. Stress	f(z)	Increment Length	Increment Diameter	Increment Side Friction	Total Ultimate Side Friction
0.7	30	0.2	120	0.90	0.454	15	4	85.53	85.53
				1.80					
0.7	30	0.2	120	2.70	1.181	15	4	222.65	308.18
				3.60					
0.7	30	0.2	120	4.50	1.909	15	4	359.77	667.95
				5.40					
0.7	30	0.2	120	6.30	2.636	15	4	496.90	1164.84
				7.20					

Total Allowable Side Friction	Base Area	Nc	Ngamma	Nq	q	qu	qa	Limiting End Bearing	Allowable End Bearing	Total Allowable Load
	12.57									
34.21	12.57	32	0	10	1.80	26.32	8.77	8.5	8.50	141.02
123.27	12.57	32	0	10	3.60	44.32	14.77	8.5	8.50	230.08
267.18	12.57	32	0	10	5.40	62.32	20.77	8.5	8.50	373.99
465.94	12.57	32	0	10	7.20	80.32	26.77	8.5	8.50	572.75

Location: Dysart Road Bridge

Project No.: 93-1033

Computed By: KW

Date: 01/26/94

SUMMARY OF RESULTS

Pier Capacity Data:

Pier Diameter=	5	ft
FS Side Friction=	2.5	
FS End Bearing=	3	
Surf. Elevation=	1080	ft
Alpha=	0.45	

Settlement:

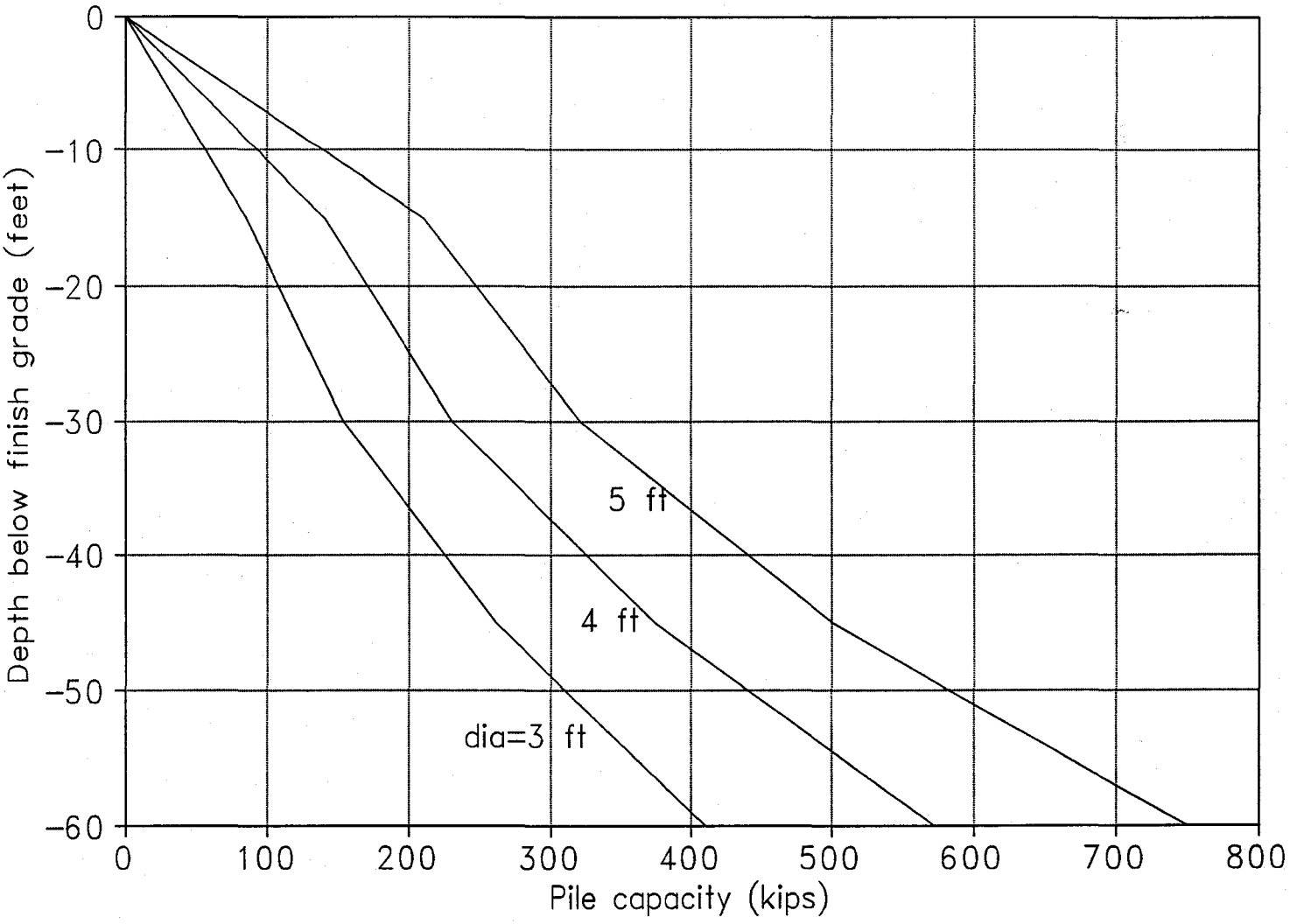
Ld Trnsfr =	0.65	
Axial Load =	350	kips
Tip Load=	-596	kips
Tip Pressure =	-30.38	ksf
Tip Modulus =	2000	ksf
Pssn's Ratio =	0.2	
Iw =	0.88	
F3 =	0.5	
Settlement =	-0.38	inches or -0.64% of diameter

NOT USED

Elevation	Depth	Soil Type	Allowable Increment Side Friction	Total Allowable Side Friction	Total Allowable End Bearing	Total Allowable Load
1080	0					
		CL-CH	42.8	42.8	166.9	209.7
1065	15					
		CL-CH	111.3	154.1	166.9	321.0
1050	30					
		CL-CH	179.9	334.0	166.9	500.9
1035	45					
		CL-CH	248.4	582.4	166.9	749.3
1020	60					

K	Phi	Su(ksf)	Unit Weight (pcf)	Eff. Stress	f(z)	Increment Length	Increment Diameter	Increment Side Friction	Total Ultimate Side Friction
0.7	30	0.2	120	0.90	0.454	15	5	106.91	106.91
				1.80					
0.7	30	0.2	120	2.70	1.181	15	5	278.31	385.22
				3.60					
0.7	30	0.2	120	4.50	1.909	15	5	449.72	834.94
				5.40					
0.7	30	0.2	120	6.30	2.636	15	5	621.12	1456.06
				7.20					

Total Allowable Side Friction	Base Area	Nc	Ngamma	Nq	q	qu	qa	Limiting End Bearing	Allowable End Bearing	Total Allowable Load
	19.63									
42.76	19.63	32	0	10	1.80	26.32	8.77	8.5	8.50	209.66
154.09	19.63	32	0	10	3.60	44.32	14.77	8.5	8.50	320.98
333.97	19.63	32	0	10	5.40	62.32	20.77	8.5	8.50	500.87
582.42	19.63	32	0	10	7.20	80.32	26.77	8.5	8.50	749.32


0
-10
-20
-30
-40
-50
-60
Depth below finish grade (feet)
0
100
200
300
400
500
600
700
800
Pile capacity (kips)
5 ft
4 ft
dia=3 ft

Location: Dysart Road Bridge

Project No.: 93-1033

Computed By: KW

Date: 01/26/94

SUMMARY OF RESULTS

Pier Capacity Data:

Pier Diameter=	4	ft
FS Side Friction=	2.5	
FS End Bearing=	3	
Surf. Elevation=	1080	ft
Alpha=	0.45	

CHECK SETTLEMENT

Settlement:

Ld Trnsfr =	0.58	from Reese & O'Neill (1989), Fig. 2 and s/d
Axial Load =	300	kips
Tip Load=	20	kips
Tip Pressure =	1.58	ksf
Tip Modulus =	288	ksf
Pssn's Ratio =	0.3	
Iw =	0.88	
F3 =	0.5	
Settlement =	0.11	inches or 0.22% of diameter

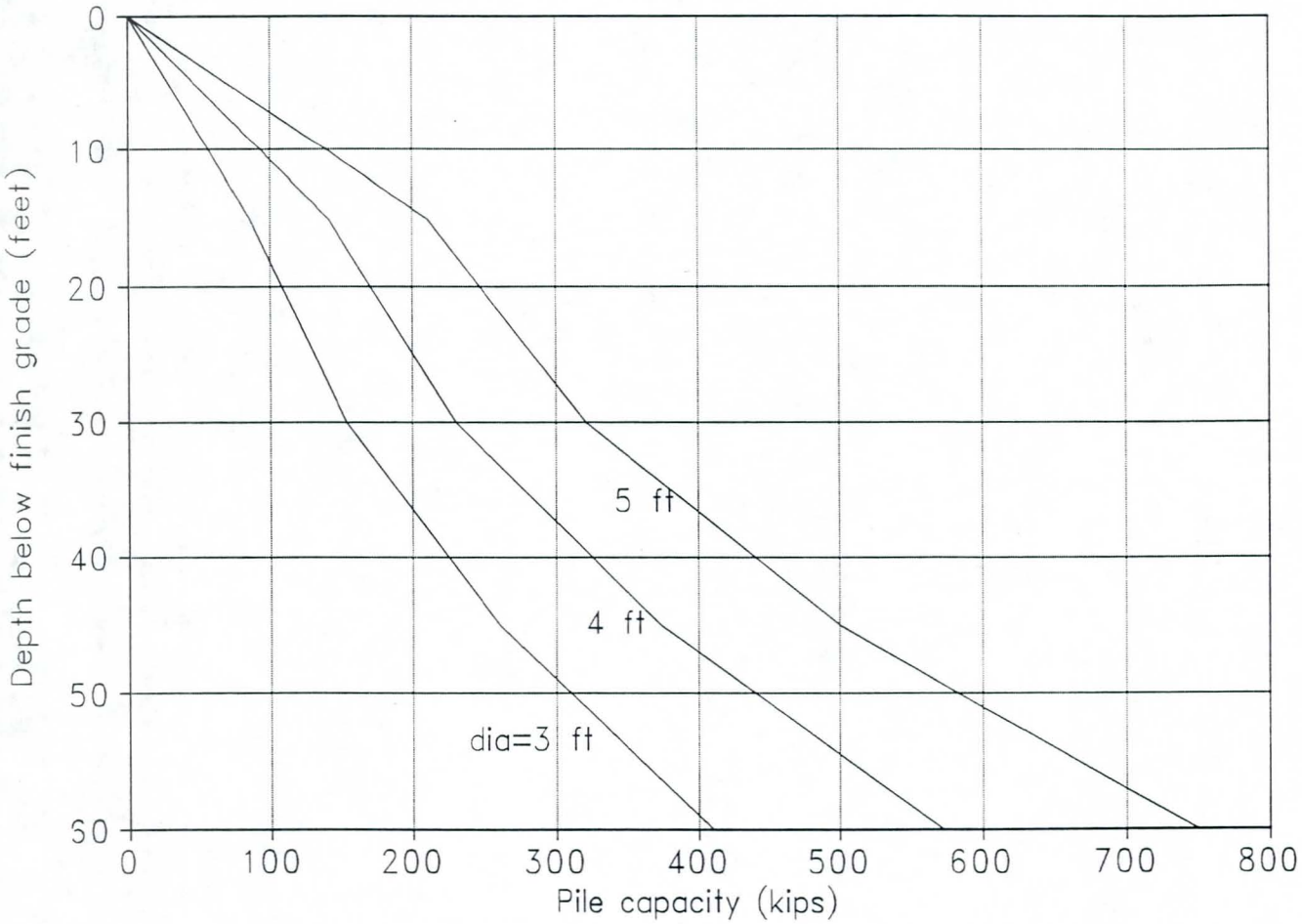
Elevation	Depth	Soil Type	Allowable Increment Side Friction	Total Allowable Side Friction	Total Allowable End Bearing	Total Allowable Load
1080	0					
		CL-CH	15.3	15.3	82.6	97.9
1070.5	9.5					
		CL-CH	37.3	52.6	106.8	159.4
1061	19					
		CL-CH	59.3	111.9	106.8	218.7
1051.5	28.5					
		CL-CH	81.3	193.2	106.8	300.0
1042	38					

K	Phi	Su(ksf)	Unit Weight (pcf)	Eff. Stress	f(z)	Increment Length	Increment Diameter	Increment Side Friction	Total Ultimate Side Friction
0.7	30	0.2	120	0.57	0.320	9.5	4	38.25	38.25
				1.14					
0.7	30	0.2	120	1.71	0.781	9.5	4	93.25	131.49
				2.28					
0.7	30	0.2	120	2.85	1.242	9.5	4	148.25	279.74
				3.42					
0.7	30	0.2	120	3.99	1.703	9.5	4	203.25	482.99
				4.56					

Total Allowable Side Friction	Base Area	Nc	Ngamma	Nq	q	qu	qa	Limiting End Bearing	Allowable End Bearing	Total Allowable Load
	12.57									
15.30	12.57	32	0	10	1.14	19.72	6.57	8.5	6.57	97.90
52.60	12.57	32	0	10	2.28	31.12	10.37	8.5	8.50	159.41
111.90	12.57	32	0	10	3.42	42.52	14.17	8.5	8.50	218.71
193.20	12.57	32	0	10	4.56	53.92	17.97	8.5	8.50	300.01

APPENDIX D

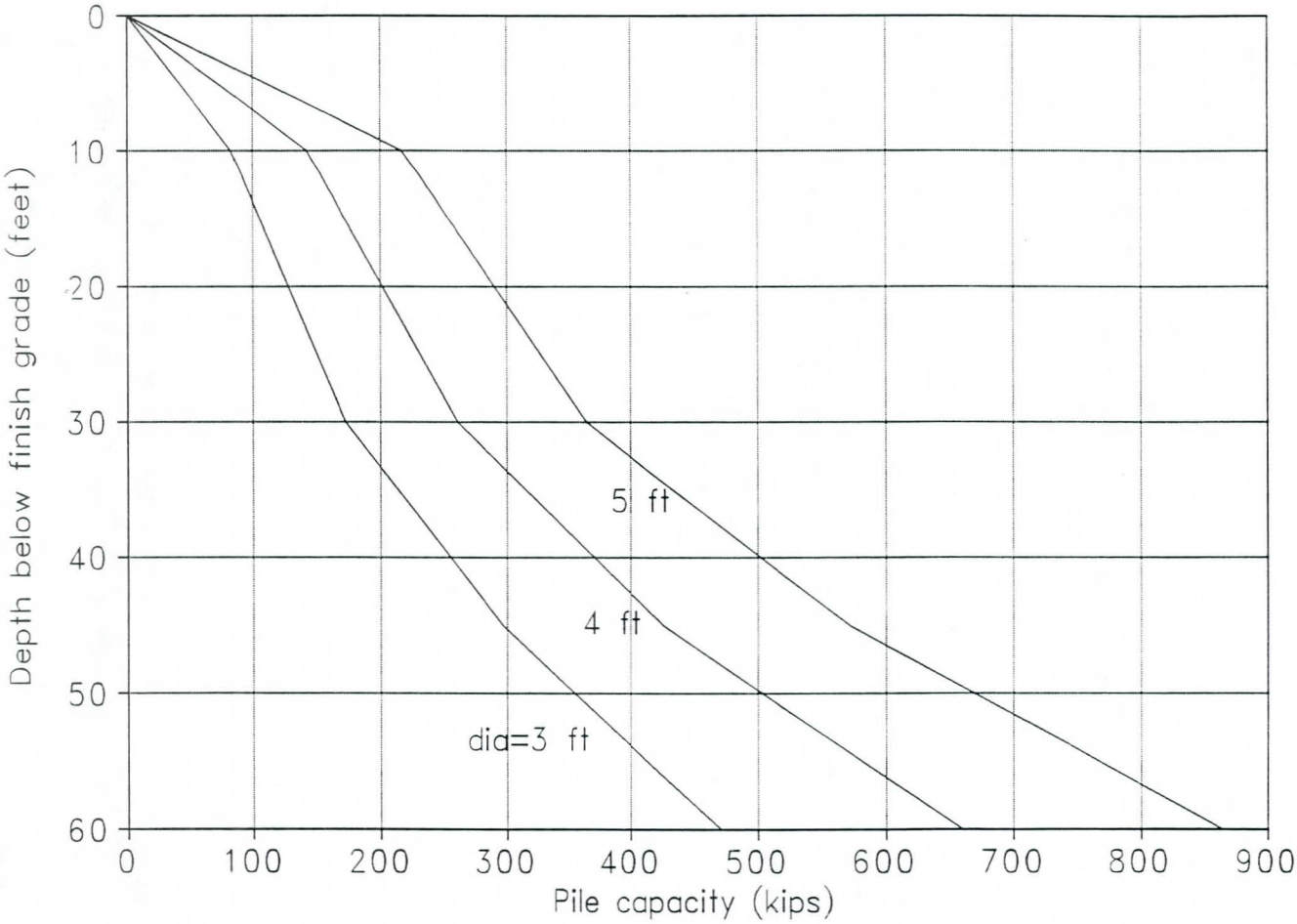
DRILLED SHAFT CAPACITIES



Drilled Shaft Capacity Chart
Dysart Road

Project No. 93-1033

THOMAS-HARTIG & ASSOCIATES, INC.



Drilled Shaft Capacity Chart
El Mirage Road

Project No. 93-1033

THOMAS-HARTIG & ASSOCIATES, INC.