Goodson & Associates, Inc.
Engineering & Cellular Concrete

GEOTECHNICAL ENGINEERING STUDY

FOR FINAL DESIGN PHASE

PROPOSED COLORADO STATE HIGHWAY 96A

4TH STREET BRIDGE

OVER THE ARKANSAS RIVER,

PUEBLO, COLORADO

CDOT PROJECT NO. STA 0961-008/13141

JANUARY 25, 2007

GAI PROJECT NO. 65569.01

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4^{TH} STREET BRIDGE

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CDOT PROJECT NO. STA 0961-008/13141

Prepared for:

Colorado Department of Transportation Region 2

As a Subconsultant to:

Figg Bridge Engineers, Inc.
1873 South Bellaire Street
Suite 1500
Denver, Colorado 80222

Attn: Mr. Steve Fultz, P.E.

Prepared by:

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Project No. 65569.01
January 25, 2007

Goodson & Associates, Inc.
Engineering & Cellular Concrete

January 25, 2007

Figg Bridge Engineers, Inc.
1873 South Bellaire Street
Suite 1500
Denver, Colorado 80222

Attn: Mr. Steve Fultz, P.E.

Re: **Geotechnical Engineering Study for Final Design Phase, Proposed Colorado State Highway 96A (4th Street) Bridge Over the Arkansas River, Pueblo, Colorado, CDOT Project No. STA 0961-008/13141**
GAI Project No. 65569.01

Goodson & Associates, Inc. (GAI) has completed a geotechnical engineering study for the Final Design Phase for the proposed Colorado State Highway 96A (4th Street) Bridge over the Arkansas River in Pueblo, Colorado. This study was performed for Colorado Department of Transportation Region 2 in general accordance with the subconsultant agreement between Figg Bridge Engineers, Inc. and GAI dated March 27, 2001, the associated Scope of Geotechnical Engineering Services dated November 9, 2005, and the Task Order No. 6 authorization letter dated January 16, 2006.

The accompanying geotechnical engineering study report presents our findings, commentary and recommendations regarding the design and construction of foundations and other earth related activities for use in the final engineering project design phase of the proposed bridge.

We appreciate the opportunity to be of service to you on this phase of your project. If you have any questions concerning this report, or if we may be of further service to you, please contact us at your convenience.

Sincerely,
GOODSON & ASSOCIATES, INC.

David M. Nasiatka, P.E.
Geotechnical Engineer

COLORADO REGISTERED PROFESSIONAL ENGINEER DAVID M. NASIATKA 25071 1.25.2007 DN

Geotechnical Engineering Study
SH 96A (4th Street) Bridge
Pueblo, Colorado
January 25, 2007

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INTRODUCTION

This report contains the results of Goodson & Associates, Inc. geotechnical engineering exploration for the proposed Colorado State Highway 96A (4^{th} Street) Bridge Over the Arkansas River, to be located in Pueblo, Colorado. The site is located in the northwest quarter of Section 36, Township 20 South, Range 65 West of the 6th Principal Meridian.

The purpose of these services is to provide information and geotechnical engineering recommendations relative to:

- subsurface soil and bedrock conditions
- groundwater conditions
- foundation design and construction
- lateral earth pressures
- pavement thickness
- earthwork
- drainage
- corrosion protection considerations

The conclusions and recommendations contained in this report are based upon the results of a site reconnaissance by a geotechnical engineer, field exploration, laboratory testing, engineering evaluation, information provided by the other project design team members, experience with similar soil conditions and structures, and our understanding of the proposed project. In addition, engineering geologic investigation and slope stability evaluation for the final design phase was performed by Michael W. West & Associates, Inc. in the area of the steep slopes at the proposed southwest abutment location. Their findings are presented in Appendix D of this report.

PROPOSED CONSTRUCTION

Based on information provided by Figg Bridge Engineers, it is understood that the bridge consists of two separate parallel bridge structures, one each for the westbound and eastbound lanes. Each bridge structure consists of Abutment 1, bridge piers 2 through 5, and Abutment 6, as shown on the attached site map.

Foundations will be drilled shafts as selected during the preliminary design phase. Information from Figg Bridge Engineers indicates that proposed drilled shafts will be 4 to 8 feet in diameter.

The overall length of the proposed improvements is from the 4th Street/Corona Avenue intersection (approximately Station 13+15) to the 4th Street/Midtown Circle Drive intersection (approximately Station 41+97).

It is understood that new fill will be placed for the approach roadway areas adjacent to both abutments. Fill thicknesses up to approximately 12 feet are anticipated for the eastbound approach roadway adjacent to Abutment 1 (southwest abutment). Fill thicknesses up to approximately 16 feet are anticipated in the vicinity of the approach roadway adjacent to Abutment No. 6 (northeast abutment).

Cut depths of up to approximately 15 feet are anticipated in the area of the westbound approach roadway lanes at the southwest abutment (Abutment 1).

SITE EXPLORATION

The scope of the services performed for this project included site reconnaissance by a geotechnical engineer, a subsurface exploration program, laboratory testing and engineering evaluation.

Field Exploration: A total of 20 exploratory borings were drilled between March 27 and August 17, 2006 to depths of approximately 10 to 135 feet at the locations shown on the Site Plan, Appendix A. The borings were advanced with a truck-mounted drilling rig, utilizing 4-1/4 inch I.D. hollow stem auger, 4 inch solid stem auger, and HX rock coring with 2.4 inch I.D. HQ drill bits.

The borings were located in the field by the project surveyors. Elevations of the borings were provided by the project surveyors. The accuracy of the boring locations and elevations should only be assumed to the level implied by the methods used.

Continuous lithologic logs of each boring were recorded during the drilling operations. At selected intervals, samples of the subsurface materials were taken by driving California barrel and/or split spoon samplers. Penetration resistance measurements were obtained by driving the

California barrel and split spoon samplers into the subsurface materials with a 140-pound hammer falling 30 inches. The penetration resistance value is a useful index to the consistency, relative density or hardness of the materials encountered. Representative bulk samples of subsurface materials were also obtained. Rock core samples were obtained in the bedrock during the rock coring activities. The penetration resistance values, rock core recovery percentages, and the Rock Quality Designation (RQD) are presented on the Boring Logs, Appendix A.

Groundwater measurements were made in each boring at the time of site exploration and at 1 to 113 days after drilling. Due to safety considerations, the borings which were drilled through existing pavements and the borings drilled in the railyard were backfilled immediately after completion of the drilling activities and patched with asphalt as necessary. Standpipe piezometers were installed in Borings BR-1, BR-2, BR-3, BR-6, BR-7, DC-1, DF-2, DF-3, DF-4 and DF-5 to facilitate future groundwater level measurements.

Laboratory Testing: The samples retrieved during the field exploration were returned to the laboratory for observation. At that time, the field descriptions were confirmed or modified as necessary and an applicable laboratory testing program was conducted to determine engineering properties of the subsurface materials. Boring logs were prepared and are presented in Appendix A.

Selected soil and bedrock samples were tested for the following engineering properties:

- Moisture content
- Dry density
- Unconfined compressive strength
- Expansive potential
- R-Value
- Grain size
- Plasticity Index
- Electrical resistivity
- Water soluble sulfate content
- pH
- Chloride content
- Angle of Internal Friction
- Splitting Tensile Strength

Laboratory test results are presented in Appendix B, and were used for the geotechnical engineering evaluations, and the development of foundation, pavement and earthwork conclusions and recommendations. The laboratory tests were performed in general accordance with the applicable ASTM, AASHTO, CDOT, or other locally accepted standards.

SITE CONDITIONS

The site currently consists of the existing SH 96A (4th Street) Bridge over the Arkansas River. The bridge also crosses over the Union Pacific Railroad (UPRR) and Burlington Northern Santa Fe (BNSF) railyard, access roads, a city street, and a flood control levee. The existing bridge is approximately 1070 feet long and 4 lanes wide. Support for the bridge consists of 2 abutments and 6 bridge piers. The Arkansas River flows approximately from northwest to southeast at the bridge location. A few feet of flowing water were present in the river at the time of the drilling activities. A flood control levee was located along the northeast side of the river. The southwest face of the levee was concrete lined. The ground surface to the northeast of the levee had a slight slope generally down to the southeast. On the southwest side of the river, the ground surface sloped steeply down to the northeast. Existing site development consisted of a shopping mall near the northeast end of the bridge. Railroad yards were present to the northwest, southeast, and under the bridge. Residential and commercial properties were located near the southwest end of the bridge. Vegetation (where present) consisted primarily of native grasses and trees. Bedrock outcrops consisting primarily of claystone shale and limestone were observed occasionally along the steep slopes on the southwest side of the Arkansas River.

SUBSURFACE CONDITIONS

Geology: Surficial geologic conditions at the site, as mapped by the U.S. Geological Survey (USGS) ([1]Scott, 1964), consist of Post-Piney Creek Alluvium of Upper Recent (Pleistocene) age. These materials generally consist of poorly sorted cobbles, pebbles, sand, and silt forming floodplain and lowest terrace deposits along major streams. Thickness of this unit is generally less than 10 feet.

Bedrock underlying the surface unit, as mapped by the USGS, consists of the Lower Transition Member of the Pierre Shale and the Smoky Hill Shale Member of the Niobrara Formation, both of Upper Cretaceous age. The Pierre Shale in this area is described as light brown bentonitic shaly chalk and light olive gray bentonitic calcareous platy shale. The Niobrara Formation in this area is described as olive black blocky ledge forming chalk, olive gray gypsiferous bentonitic calcareous shale, and yellowish gray platy chalk. Thicknesses of these bedrock materials are reported to be on the order of approximately 200 to 300 feet each.

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[1]Scott, Glenn R., 1964, ***Geology of the Northwest and Northeast Pueblo Quadrangles, Colorado***, United States Geological Survey, Map I-408.

Mapping completed by the Colorado Geological Survey (CGS) ([2]Hart, 1974), indicates the site is in an area of "Low Swell Potential." Potentially expansive materials mapped in this area include several bedrock formations and many surficial deposits.

The information presented in these USGS and CGS publications is of a generalized nature and local variations are possible. A more detailed description of the general surficial geologic conditions is presented in the engineering geologic investigation report prepared by Michael W. West and Associates, Inc. dated October 31, 2006, Project No. 05675, presented in Appendix D.

Soil and Bedrock Conditions: In the bridge borings (BR-1 through BR-8) existing asphalt pavement, sand fill, or natural gravelly sand were typically encountered at the ground surface. These materials were typically underlain by clay fill or natural silty sand. Claystone was encountered below these materials in Borings BR-1 and BR-2 (at the proposed southwest bridge abutment location) at depths of approximately 8-1/2 to 11-1/2 feet. The claystone was underlain by very hard shale bedrock at depths of approximately 22 to 31 feet. The shale bedrock continued to the maximum depths explored in these two borings, approximately 108 to 135 feet.

In the other bridge borings (BR-3 through BR-8) the sand fill, clay fill, and natural sand soils were underlain by the shale bedrock at depths of approximately 8 to 45 feet. The shale bedrock continued to the maximum depths explored in these borings, approximately 75 to 103 feet.

In the deep cut and deep fill borings drilled south of the proposed southwest abutment (Borings DC-1 and DF-1), natural clay and sand soils were encountered at the ground surface. These soils were underlain by claystone at approximately depths of 3 to 10-1/2 feet. The claystone continued to the maximum depth explored in Boring DC-1, approximately 30 feet. In Boring DF-1, the claystone was underlain by the shale bedrock at an approximate depth of 22 feet. The shale bedrock continued to the maximum depth explored, approximately 24 feet.

In the deep fill borings drilled east of the proposed east abutment (DF-2 through DF-5), existing asphalt pavement, sand fill, or natural sand or clay soils were typically encountered at the ground surface. These materials were typically underlain by natural gravelly and silty sands. The gravelly sand continued to the maximum depth explored in Boring DF-3, approximately 25 feet. In the remaining deep fill borings, the natural sands were underlain by the shale bedrock which

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[2]Hart, Stephen S., 1974, ***Potentially Swelling Soil and Rock in the Front Range Urban Corridor, Colorado***, Colorado Geological Survey, Environmental Geology No. 7.

was encountered at depths of approximately 30 to 34 feet. The shale bedrock continued to the maximum depths explored, approximately 31 to 40 feet.

In the pavement borings (P-1 through P-6), existing asphalt pavement or clay fill was encountered at the ground surface. The clay fill continued to the maximum depth explored in Boring P-3, approximately 11 feet. In Boring P-2, the clay fill was underlain by claystone at approximately 4 feet, with the claystone continuing to the maximum depth explored, approximately 10 feet. In Borings P-1, P-4, P-5 and P-6, sand fill and natural sands were encountered below the existing asphalt. The natural sands continued to the maximum depths explored, approximately 11 feet.

In addition to the exploratory borings, 2 exploratory trench pits (ET-2 and ET-3) were excavated in the vicinity of the proposed southwest abutment. During previous drilling activities in 2001 in this area, large concrete debris and other existing fill materials were suspected by the drilling personnel. The soil conditions encountered in these 2 pits were as follows:

ET-2

Approximate Depth Below Existing Ground Surface	Soil Description
0 feet to 7-1/2 feet	Fill, Sand, silty, gravelly, occasionally clayey, dry to slightly moist, brown to light brown with occasional zones of concrete debris, appeared to be old slabs, and curb and gutter fragments, up to approximately 4 foot by 4 foot by 1 foot size, also asphalt fragments
7-1/2 feet to 8 feet	Old asphalt pavement
8 feet to 8.2 feet	Old aggregate base course
8.2 feet to 16 feet	Fill, Sand, clayey, gravelly, slightly moist, brown to light brown, with occasional claystone fragments, shale fragments, and cobbles
16 feet to 17 feet	Claystone and shale, brown and occasional gray, occasionally calcareous

ET-3

Approximate Depth Below Existing Ground Surface	Soil Description
0 feet to 4-1/2 feet	Fill, Sand, silty, clayey, with gravel, cobbles, asphalt fragments, concrete fragments, sparse metal and wire, dry to slightly moist, brown to light brown
4-1/2 feet to 5 feet	Old asphalt pavement and concrete pavement
5 feet to 9 feet	Fill, Sand, silty and Clay, sandy, with gravel and cobbles, slightly moist, brown to light brown
9 feet to 10 feet	Clay and weathered claystone, sandy, slightly moist, brown and orange brown

No groundwater was encountered in either ET-2 or ET-3 at the time of excavating. The exploratory trench pits were backfilled immediately after completion of the field activities.

Field test results during the drilling activities indicated that the clay fill was stiff to very stiff in consistency and that the natural sandy clay soil was soft to stiff. The sand fill varied from very loose to dense in relative density and the natural sand soils varied from very loose to very dense. The claystone varied from firm to very hard and the shale bedrock is typically very hard.

A more complete description of the subsurface conditions encountered in the borings can be found in the Boring Logs, Legend and Notes, Appendix A.

Groundwater Conditions: Groundwater was encountered at depths ranging from approximately 1-1/2 to 30 feet in some of the exploratory borings at the time of field exploration. When checked 1 to 113 days after drilling, groundwater was measured at depths in the range of approximately 0 to 41 feet. These observations represent only current groundwater conditions, and may not be indicative of other times, or at other locations. Groundwater levels can be expected to fluctuate with varying seasonal and weather conditions.

Zones of perched and/or trapped groundwater may occur at times in the subsurface soils overlying bedrock, on top of the bedrock surface or within permeable fractures in the bedrock materials. The location and amount of perched water is dependent upon several factors, including hydrologic conditions, type of site development, irrigation demands on or adjacent to the site, fluctuations in water features such as the Arkansas River, and seasonal and weather conditions.

A groundwater level observation plan can be implemented to provide information regarding fluctuations in groundwater levels at the project site. Such a program should include installation of groundwater observation standpipe piezometers, and periodic measurement of groundwater levels over a sufficient period of time. The possibility of groundwater fluctuations should be considered when developing design and construction plans for the project.

CONCLUSIONS AND RECOMMENDATIONS

General Geotechnical Considerations: During the preliminary design phase, a drilled shaft foundation was chosen as the preferred foundation alternative. Based on the exploratory borings drilled during this final design phase of the project, the soil and bedrock conditions appear to be suitable for this type of foundation system. Existing fill materials will require particular attention in the design and construction of the proposed bridge.

Drilled shafts into bedrock transmit structural loads to a stratum of high bearing capacity. Soil and groundwater conditions indicate that temporary steel casing will likely be required to properly drill and clean drilled shaft holes prior to concrete placement. In addition, significant dewatering activities will be necessary for construction of any foundation elements in or adjacent to the Arkansas River.

Large concrete debris and other existing fill materials were encountered in the vicinity of the proposed southwest abutment. These materials appear to not have been properly compacted for support of the anticipated new embankment fill for the southwest approach roadway. Consideration should be given to removing and properly recompacting these materials during construction.

Additional commentary and recommendations for foundation systems, pavements, and other earth connected phases of the project are presented below.

Drilled Shaft Foundations: Drilled shafts (drilled piers) can be considered for support of the proposed SH 96A (4^{th} Street) Bridge. The drilled shafts should penetrate well into the very hard shale bedrock. The AASHTO Load and Resistance Factor Design (LRFD) Bridge Design

Specifications, Interim 2006 Edition, indicates that axial compression loads on a drilled shaft socketed into rock is carried solely in shaft side resistance until a total shaft movement on the order of 0.4 inches occurs.

For the Load and Resistance Factor Design (LRFD) method, an ultimate side shear of 21 ksf for the portion of the drilled shaft in the very hard shale bedrock, and a resistance factor of 0.55 may be used for design purposes for the foundation elements at Abutment 1, Pier 5 and Abutment 6. At Pier 2, for the LRFD method, an ultimate side shear of 12 ksf for the portion of the drilled shaft in the very hard shale bedrock, and a resistance factor of 0.55 may be used. At Piers 3 and 4, for the LRFD method, an ultimate side shear of 17 ksf for the portion of the drilled shaft in the very hard shale bedrock, and a resistance factor of 0.55 may be used. No end bearing resistance should be assigned to the drilled shaft design based on the information presented in the Interim 2006 LRFD document.

Based on ground surface elevations provided by the project surveyors and the depths to bedrock observed in the borings, elevations of the top of the very hard shale bedrock were as follows:

Boring No.	Approximate Top of Very Hard Shale Bedrock
BR-1	4702 Feet
BR-2	4690 Feet
BR-3	4655 Feet
BR-4	4649 Feet
BR-5	4647 Feet
BR-6	4646 Feet
BR-7	4643 Feet
BR-8	4643 Feet

The drilled shafts should penetrate a minimum of 25 feet into the very hard shale bedrock. Based on a 25 foot minimum penetration, the bedrock conditions encountered in the exploratory borings, the laboratory test results, and load information from Figg Bridge Engineers, the following bottom of shaft elevations are recommended:

Boring No.	Approximate Bottom of Drilled Shaft Elevation
BR-1	4660 Feet
BR-2	4660 Feet
BR-3	4621 Feet
BR-4	4610 Feet
BR-5	4607 Feet
BR-6	4615 Feet
BR-7	4618 Feet
BR-8	4618 Feet

Laboratory testing indicated that some of the claystone in the vicinity of the southwest abutment has expansive potential. Required drilled shaft penetration should be balanced against potential uplift due to possible expansion of the claystone on the site.

For design purposes, the uplift force on each shaft can be determined on the basis of the following equation:

$$U_p = 5 \: x \: D$$

Where: U_p = the uplift force in kips, and
D = the pier diameter in feet

Uplift forces on shafts should be resisted by a combination of dead-load and skin friction from shaft penetration below a depth of 18 feet and in the very hard shale bedrock.

The drilled shafts should be reinforced full depth for the applied axial, lateral and uplift stresses imposed. The amount of reinforcing steel for expansion should be determined by the tensile force created by the uplift force on each shaft, with allowance for deadload. A minimum 4-inch void space should be provided beneath grade beams and drilled shaft caps, if used.

For lateral behavior of drilled shafts, "L-Pile" analysis can be used for evaluation. For final design phase purposes, recommended "L-Pile" parameters are presented below. The "L-Pile" parameters are based on information presented in "COM624P-Laterally Loaded Pile Analysis Program, Version 2.0", by Shin-Tower Wang and Lymon C. Reese, published by Federal Highway Administration, Publication No. FHWA-SA-91-048, dated June 1993.

For the West Abutment (Abutment 1), and at Bridge Piers 2, 3, 4, and 5:

Soil or Bedrock Type	E_{50}-Value	k_s-Value	Cohesion, c	Unit Weight, γ	Angle of Internal Friction, φ
Fill – Sand	--	25 pci (above G.W.T.) 20 pci (submerged)	--	100 pcf (above G.W.T.) 35 pcf (submerged)	30°
Fill – Clay	0.010	--	500 psf	100 pcf	0°
Natural Silty Sand to Gravelly Sand	--	90 pci (above G.W.T.) 60 pci (submerged)	--	120 pcf (above G.W.T.) 60 pcf (submerged)	33°
Firm to Very Hard Claystone Bedrock	0.005	--	4500 psf	115 pcf	0°
Very Hard Shale Bedrock	0.004	--	6000 psf *	125 pcf *	0°

* **Note:** These values represent low conservative cohesion and unit weight properties for the shale bedrock. Alternatively actual unconfined compressive strength laboratory test results can be considered for lateral load design.

For the East Abutment (Abutment 6):

Soil or Bedrock Type	E_{50}-Value	k_s-Value	Cohesion, c	Unit Weight, γ	Angle of Internal Friction, φ
Fill - Sand	--	25 pci	--	100 pcf	30°
Fill - Clay	0.007	--	700 psf	100 pcf	0°
Natural Silty Sand to Gravelly Sand	--	90 pci (above G.W.T.) 60 pci (submerged)	--	125 pcf (above G.W.T.) 65 pcf (submerged)	33°
Very Hard Bedrock	0.004	--	6000 psf *	125 pcf *	0°

* **Note:** These values represent low conservative cohesion and unit weight properties for the shale bedrock. Alternatively actual unconfined compressive strength laboratory test results can be considered for lateral load design.

The AASHTO LRFD literature indicates that drilled shafts should be considered to work in group action for axial and tensile loadings if the horizontal spacing is less than 3 shaft diameters. For lateral loading, group action should be considered if the shaft spacing in the direction of loading is less than 5 shaft diameters. Adjacent shafts should bear at about the same elevation. The capacity of individual shafts should be reduced when considering the effects of group action. Capacity reduction is a function of shaft spacing and the number of shafts within a group. Group effects should be considered as needed using acceptable methods.

Drilling should be possible with conventional single flight power augers on the majority of the site. However, areas of very hard shale bedrock may be encountered where specialized heavy duty drilling equipment may be required. In addition, occasional relatively thin soft bentonite beds were observed in some of the very hard shale bedrock core samples obtained from the exploratory borings. If a soft bentonite bed is present at the bottom of a drilled shaft excavation, it is recommended that the drilled shaft be extended down below the bentonite bed in order that the bottom of the drilled shaft bears on the very hard shale bedrock.

Soil and groundwater conditions indicate that temporary steel casing will likely be required to properly drill and clean drilled shafts prior to concrete placement. Groundwater should be removed from each shaft hole prior to concrete placement. It is highly recommended that the drilled shaft concrete be placed immediately after completion of drilling and cleaning. If degradation of the shaft hole occurs due to time delays or other causes, then remedial measures or abandonment of the hole may be necessary. If shaft concrete cannot be placed in dry conditions or in 3 inches or less of water, then a tremie or pumping system should be used for concrete placement. Concrete used in the drilled shafts should be a fluid mix with sufficient slump to fill the void between reinforcing steel and the shaft hole. If casing is used for shaft construction, it should be withdrawn in a slow continuous manner maintaining a sufficient head of concrete to prevent infiltration of water or the creation of voids in the concrete.

The drilled shafts should be constructed in accordance with Section 503 of the CDOT Standard Specifications for Road and Bridge Construction, (2005 Edition). A geotechnical engineer should observe the installation of the drilled shafts on a full-time basis to check that the appropriate bearing stratum is penetrated and to observe other shaft construction procedures.

Lateral Earth Pressures: For final design phase purposes recommended equivalent fluid pressures for unrestrained foundation elements are:

- Active:
  Cohesive soil backfill (on-site natural sandy clay)............................50 psf/ft
  Cohesionless soil backfill (on-site existing sand fill)........................45 psf/ft
  Cohesionless soil backfill (on-site natural sand)..............................40 psf/ft
  CDOT Class 1 Structure Backfill...40 psf/ft
  On-site bedrock materials..................................not recommended for use

- Passive:
  Cohesive soil backfill (on-site natural sandy clay)..........................300 psf/ft
  Cohesionless soil backfill (on-site existing sand fill)......................350 psf/ft
  Cohesionless soil backfill (on-site natural sand)............................400 psf/ft
  CDOT Class 1 Structure Backfill...400 psf/ft

When the design includes restrained elements, the following equivalent fluid pressures are recommended:

- At Rest:
  Cohesive soil backfill (on-site natural sandy clay)..............................70 psf/ft
  Cohesionless soil backfill (on-site existing sand fill)...........................65 psf/ft
  Cohesionless soil backfill (on-site natural sand)................................60 psf/ft
  CDOT Class 1 Structure Backfill..60 psf/ft
  On-site bedrock materials...................................not recommended for use

The lateral earth pressures herein are not applicable for submerged soils. Additional recommendations will be necessary if such conditions are to be included in the design. The above values are based on horizontal backfill surfaces. Any surcharge loadings should be added to the above values.

Fill against abutments, wing walls, and retaining walls should be properly compacted. Expansive soils or claystone should not be used as backfill against abutments or walls. Compaction of each lift adjacent to walls should be accomplished carefully. Overcompaction may cause excessive lateral earth pressures which could result in wall movement.

Seismic Considerations: Based on Section 3.10 of the AASHTO LRFD Bridge Design Specifications, 2004 Edition, and based on the soil and bedrock conditions encountered in the exploratory borings, the following values can be used for final design phase purposes:

- Acceleration Coefficient, A, of 0.025. (Section 3.10.2 and Figure 3.10.2-1)

- Seismic Zone 1. (Section 3.10.4 and Table 3.10.4-1)

- Soil Profile Type I. (Section 3.10.5.2)

- Site Coefficient, S, of 1.0. (Section 3.10.5.1 and Table 3.10.5.1-1)

Compression of Existing Soils: Total settlement of the existing natural soils and existing fill soils due to the placement of new embankment fill consists of (1) immediate settlement, (2) primary consolidation settlement, and (3) secondary compression settlement. The exploratory borings drilled in the vicinity of the proposed new embankment fills encountered primarily sand soils and unsaturated clays. For these types of soils, immediate settlements are typically the primary form of settlement when the soils are subjected to new loadings. Based on these types of soils and the anticipated new embankment fill thicknesses of approximately 12 to 16 feet, settlements of approximately 1 to 5 inches are possible. The majority of the settlement should occur during the construction phase.

In the vicinity of the southwest abutment approach roadway, existing fill was encountered in exploratory trench pits ET-2 and ET-3. The fill contained large concrete fragments and other construction debris. It is highly recommended that a proof rolling be accomplished on this existing fill prior to the placement of any new embankment fill. Areas which deform or show other signs of instability during the proof roll should be overexcavated, removed and replaced with properly compacted non-expansive fill.

Compression of New Embankment Fill: The amount of compression of embankment fill materials is primarily dependent on the height of the fill and on the type of fill soils. For sand or gravel soils, the majority of the compression would take place during construction. For fill soils with moderate to high clay or silt contents, some of the compression will occur after the embankment has been constructed. For fill heights of approximately 12 to 16 feet, estimated compression of sand or gravel fill would be approximately 1/2 to 2 inches. For clay or silt fill, estimated compression values would be approximately 1 to 5 inches. If possible, embankments with moderate to high clay or silt content fill should be constructed prior to final grading and roadway construction to allow this compression to occur. Benchmarks should be established in order to monitor the degree of compression as it occurs. GAI recommends that additional laboratory testing be performed on the specific proposed fill materials prior to embankment construction in order to better approximate the amount of post construction compression which may occur.

Pavement Thickness Calculations: Pavement thickness calculations for the project were based on the procedures outlined in the Colorado Department of Transportation (CDOT) Roadway Design Manual, 2007 Edition. Traffic criteria was provided by PBS&J and consisted of a year 2001 Average Daily Traffic (ADT) value of 27,637 and a year 2025 forecast ADT value of 39,600. Vehicle distributions consisted of 90% passenger cars and pickups, 9% single unit trucks, and 1% combination unit trucks. A lane factor of 0.45 (4 lane design condition) was used for 18-kip equivalent single axle load (ESAL) calculations. A design life of 20 years was utilized. Based upon the above input data, 20 year 18 kip ESAL values of 3,973,456 for flexible pavement design and 4,999,398 for rigid pavement design were calculated.

R-value laboratory testing on samples obtained from the pavement borings (P-1 through P-6) indicated R-values in the range of 26 to 73. For the flexible pavement thickness design calculations, a serviceability index loss of 2.0, an overall deviation factor of 0.44, and a reliability factor of 95% were utilized. A strength coefficient of 0.44 for Hot Mix Asphalt (HMA) was used to calculate the flexible pavement thickness alternatives.

Recommended alternatives for flexible pavements are as follows:

Subgrade R-Value	Alternative	Full Depth Hot Mix Asphalt (HMA) Pavement Thickness (inches)
60	A	8
48	B	8-1/2
26	C	11

A summary of the pavement thickness calculations is presented in Appendix C.

Based on discussions with representatives of CDOT and Figg Bridge Engineers, it is understood that CDOT Region 2 Materials desires placement of 6 inches of Class 6 Aggregate Base Course (ABC) below the HMA. The desired pavement sections proposed by CDOT Region 2 Materials are as follows:

- For roadway areas where the existing subgrade has an R-Value of 48 or greater for a minimum of 2 feet below grade, the pavement section would consist of 7-1/2 inches of HMA over 6 inches of Class 6 ABC.

- For roadway areas where new fill will be placed, the fill will have an R-Value of 48 or greater and will be a minimum of 2 feet in thickness. The pavement section would consist of 7-1/2 inches of HMA over 6 inches of Class 6 ABC.

Calculations by GAI indicate that these proposed pavement thickness sections satisfy Equation 3.2 in Section 3.6 of the 2007 CDOT Pavement Design Manual based on the calculated Structural Numbers from the traffic criteria provided and based on the specified R-Values.

From approximately Station 17+80 to Station 19+20 of the SH96 westbound lanes cut depths of approximately 8 to 15 feet are proposed in order to achieve the final desired grade elevations. In this area, it is anticipated that the existing soils at the proposed pavement subgrade elevation will have an R-value less than 48 and have expansive potential. Laboratory test results of this material indicate a swell potential of approximately 4 percent with a plasticity index (PI) of 23. Section 2.3 of the 2007 CDOT Pavement Design Manual indicates that a subexcavation depth of 3 feet be accomplished for potentially expansive subgrade soils with PI values in the range of 20 to 30.

It is recommended that the top 2 feet of this subexcavated zone be replaced with a minimum R-value 48 fill material. The fill should placed and compacted in accordance with Section 203.07 of the CDOT Standard Specifications for Road and Bridge Construction, 2005 Edition.

Variation of design parameters such as design life, traffic loadings, reliability, serviceability index, and strength coefficients may change the thickness of the pavement sections presented. Goodson & Associates, Inc. is available to discuss the design parameters and their effects on pavement design and reevaluate the pavement thickness sections if needed.

The performance of the pavements can be improved by reducing excess moisture which can reach the subgrade soils, especially where expansive soils are present. Site grading at a minimum 2% grade away from the pavements is recommended. Side ditches, culvert inlets and culvert outlets should be designed to provide proper drainage away from the pavements. Appropriate erosion control measures should be taken.

Pavement subsurface drainage can be considered for areas where R-Value 48 subgrade fill materials will be placed directly upon low permeability claystone and/or shale. This situation could occur in the proposed cut areas for the southwest approach roadway. The subsurface drainage at these locations can consist of a combination of free draining gravel and perforated pipes in order to remove waters which may collect on top of the low permeability materials directly under the R-Value 48 fill. GAI can provide more specific recommendations if needed.

Earthwork: For the final design phase, the following commentary and recommendations regarding earthwork are provided for consideration.

- **Site Clearing and Subgrade Preparation:**

1. Strip and remove existing vegetation, debris, and other deleterious materials from proposed bridge structure areas. All exposed surfaces should be free of mounds and depressions which could prevent uniform compaction.

2. If unexpected fills or underground facilities are encountered during site clearing, such features should be removed prior to backfill placement and/or construction. All excavations should be observed by a geotechnical engineer prior to backfill placement.

3. Stripped materials consisting of vegetation and organic materials should be removed from the site or used to revegetate exposed slopes after completion of grading operations. If it is necessary to dispose of organic materials on-site, they should be placed in non-structural areas.

4. Sloping areas steeper than 5:1 (horizontal:vertical) should be benched to reduce the potential for slippage between existing slopes and fills. Benches should be level and wide enough to accommodate compaction and earth moving equipment.

5. Depending on the relative locations of the new bridge and the old existing foundation elements, demolition and removal of the old foundations may be necessary. If removal is not possible, then other precautions will be required. Concrete and asphalt materials derived from demolition of the existing structures should not be used in any on-site fills unless the materials are reprocessed to a maximum particle size of 3 inches. Recycled material should be inorganic and meet specific gradation requirements depending on the specific use for which it is intended.

6. All exposed areas which will receive fill, once properly cleaned and benched where necessary, should be scarified to a minimum depth of 12 inches, moisture conditioned,

and properly compacted. Proof-rolling of the subgrade should be required to determine stability prior to fill placement.

- **Excavation**:

1. It is anticipated that excavations into the on site fill and natural soils can be accomplished with conventional earthmoving equipment.

2. If excavations need to penetrate into the bedrock, ripping or jack-hammering may be needed to advance the excavation, particularly in confined excavations.

3. Depending upon depth of excavation and seasonal conditions, groundwater may be encountered in excavations on the site. Pumping from sumps may be utilized to control water within excavations. Care should be taken to avoid developing quick conditions in excavations below the groundwater level.

4. Some of the on-site soils may pump or become unstable or unworkable at high water contents. Workability may be improved by scarifying and drying. Lightweight excavation equipment may be required to reduce subgrade pumping. Overexcavation of isolated wet zones and replacement with granular materials may be necessary. The overexcavation and replacement activities should be accomplished in accordance with Sections 203 and 206 of the CDOT Standard Specifications for Road and Bridge Construction, 2005 Edition.

- **Fill Materials**:

1. Clean on-site soils or approved imported soils may be used as fill material for general site grading, foundation areas and foundation backfill.

2. On-site claystone bedrock materials are not recommended for use beneath structural areas of the site or as backfill. If claystone bedrock materials are used for general site grading, placement in fills at non-structural locations on the site is recommended.

3. Frozen soils should not be used as fill or backfill.

4. Imported granular fill soils (if required) should conform to the following or be approved by the Project Geotechnical Engineer:

Sieve Size or Number	Percent finer by weight (ASTM C136)
2"	100
No. 4 Sieve	30-100
No. 50 Sieve	10-60
No. 200 Sieve	0-20

- Liquid Limit.......35 (max)
- Plasticity Index.......6 (max)
- Maximum expansive potential (%)*....... 1.0

* Measured on a sample compacted to approximately 95 percent of the ASTM D698 (AASHTO T99) maximum dry density at about 3 percent below optimum water content. The sample is confined under a 200 psf surcharge and submerged.

- **Placement and Compaction**:

1. Place and compact fill in horizontal lifts, using equipment and procedures that will produce the required moisture contents and densities throughout the lift.

2. No fill should be placed over frozen ground.

3. Materials should be compacted in accordance with Section 203.07 of the CDOT Standard Specifications for Road and Bridge Construction, 2005 Edition.

4. If a well defined maximum density curve cannot be generated by impact compaction in the laboratory for any fill type, fill should be compacted to a minimum of 80 percent relative density as determined by ASTM D4253 and D4254.

- **Compliance**: Performance of foundations and pavement elements supported on compacted fills or prepared subgrade depend upon compliance with "Earthwork" recommendations. To assess compliance, observation and testing should be performed under the direction of a geotechnical engineer.

- **Excavation and Trench Construction**: Excavations into the on-site soils and bedrock will likely encounter a variety of conditions, including possible caving and/or groundwater. The individual contractor(s) should be made responsible for designing and constructing stable, temporary excavations as required to maintain stability of both the excavation sides and bottom. All excavations should be sloped or shored in the interest of safety following local and federal regulations, including current OSHA excavation and trench safety standards.

  The soils and bedrock penetrated by excavations may vary significantly across the site. The contractor should verify the conditions which exist throughout the proposed area of excavation. The conditions should be evaluated to determine any excavation modifications necessary to maintain safe conditions.
  All vehicles and soil piles should be a minimum lateral distance from the crest of the slope equal to the slope height. The exposed slope face should be protected against the elements.

Drainage: For the final design phase, the following commentary and recommendations regarding drainage are provided.

- **Surface Drainage**: Positive drainage should be provided during construction and maintained throughout the life of the proposed bridge and roadway. Infiltration of water into utility or foundation excavations should be prevented during construction. Backfill against walls and in utility trenches should be well compacted and free of all construction debris to reduce the possibility of moisture infiltration.

- **Subsurface Drainage**: In order to reduce the potential for hydrostatic forces to develop against abutment walls, wing walls, and retaining walls, it is recommended that vertical drainage against the retained earth side of the walls be provided. Either a free draining gravel zone (minimum 1 foot width) protected from the wall backfill with drainage geofabric, or prefabricated drainage panels (such as Miradrain 6000 or equivalent) should be used. The free draining gravel (if used) should meet the requirements of Class B or C

Filter Material as presented in Section 703.09 of the CDOT Standard Specifications for Road and Bridge Construction, 2005 Edition. The vertical drainage gravel or drainage panels should discharge into perforated drain lines (placed at the bottom of the walls) and should extend to within 2 feet of the finished surface grade. An impervious soil should be used in the upper layer of backfill to reduce the potential for water infiltration.

The perforated drain lines should be embedded in Class B or C filter material (or drainage geofabric) and should be provided with adequate clean-outs for periodic maintenance. The perforated drain lines should be sloped to daylight outfalls. Animal guards should be considered at the pipe outfalls.

Corrosion Protection Considerations: Results of water soluble sulfate testing on selected soil and bedrock samples obtained from the exploratory borings were in the range of approximately 0.030 to 0.232 percent. Previous studies indicated water soluble sulfate concentrations as high as 1.449 percent. Based on these test results, Class 2 concrete as presented in Table 601-4 of the CDOT Standard Special Provisions to the Standard Specifications for Road and Bridge Construction should be used for concrete which will be in contact with the on site soils or bedrock.

Laboratory test results on selected samples indicate electrical resistivities ranging from approximately 200 to 1,110 ohm-centimeters, pH values ranging from 4.6 to 8.2, and chloride contents ranging from approximately 0.00002 to 0.205 percent. These values can be used to determine potential corrosive characteristics of the on-site soils and bedrock with respect to contact with the various underground materials which will be used for project construction.

GENERAL COMMENTS

It is recommended that the Geotechnical Engineer be retained to provide a general review of final design plans and specifications in order to confirm that grading and foundation recommendations have been interpreted and implemented. In the event that any changes of the proposed project are planned, the conclusions and recommendations contained in this report should be reviewed and the report modified or supplemented as necessary.

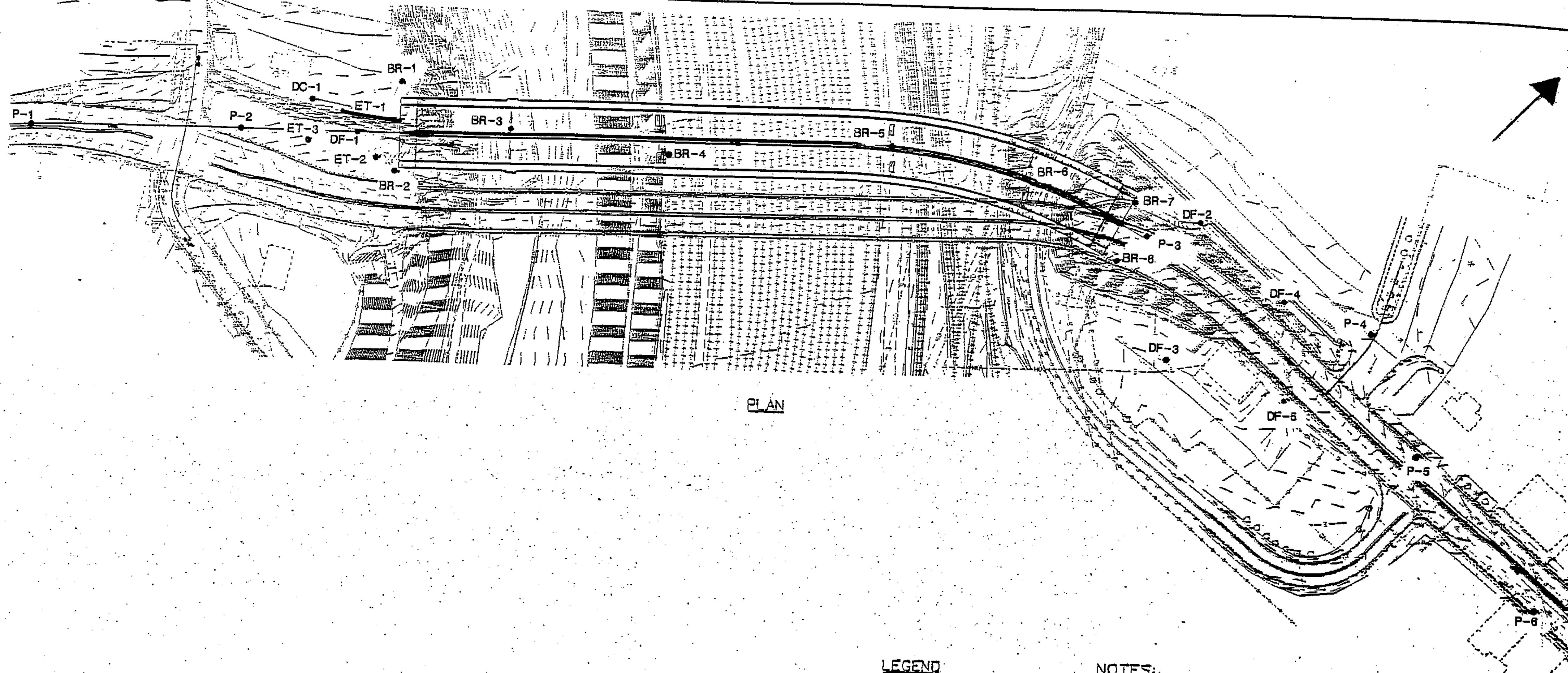
The Geotechnical Engineer should also be retained to provide services during excavation, grading, foundation and construction phases of the work. Observation of foundation excavations should be performed prior to placement of reinforcing and concrete to confirm that satisfactory bearing materials are present and is considered a necessary part of continuing geotechnical engineering services for the project. Construction testing, including field and laboratory evaluation of fill, backfill, pavement materials, concrete and steel should be performed to determine whether applicable project requirements have been met.

The analyses and recommendations in this report are based in part upon data obtained from the field exploration. The nature and extent of variations beyond the location of the exploratory borings may not become evident until construction. If variations appear evident, it may be necessary to re-evaluate the recommendations of this report.

Our professional services were performed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical engineers practicing in this or similar localities. No warranty, express or implied, is made. We have prepared the report as an aid in design of the proposed project. This report is not a bidding document. Any contractor reviewing this report must draw his or her own conclusions regarding site conditions and specific construction techniques to be used on this project.

This report is for the exclusive purpose of providing geotechnical engineering and/or testing information and recommendations. The scope of services for this project does not include, either specifically or by implication, any environmental assessment of the site or identification of contaminated or hazardous materials or conditions. If the owner is concerned about the potential for such contamination, other studies should be undertaken.

APPENDIX A



PLAN

LEGEND

BR - Bridge Boring

DC - Boring in Future Cut Area

DF - Boring in Future Fill Area

P - Pavement Boring

NOTES:

1. Exploratory borings were drilled between March 27 and August 17, 2006, with 4 1/4-inch I.D. continuous flight hollow stem power auger, 4-inch solid stem power auger, and HX rock coring with 2.4-inch I.D. HQ drill bit.
2. Locations of the exploratory borings were determined by the project surveyors after the drilling activities were completed.
3. Elevations of the exploratory borings were provided by the project surveyors. The logs of the exploratory borings are drawn to depth.
4. The exploratory boring locations should be considered accurate only to the degree implied by the method used.
5. The boundary lines between materials shown on the exploratory boring logs are approximate and the transitions between strata may be gradual.
6. Ground water levels shown on the logs were measured at the time and under conditions indicated. Fluctuations in the groundwater levels may occur with time.
7. For boring logs and additional information, see Geotechnical Details sheet.

SCALE: 1"=166'

PRELIMINARY NOT FOR CONSTRUCTION - OCTOBER 18, 2006 - 90% SUBMITTAL

Print Date: 10/13/2006	
Drawing File Name: 13141_Geotechnical_Plan.dgn	
Horiz. Scale:	Vert. Scale:
Unit Information	Unit Leader Initials

Sheet Revisions

Date:	Comments	Init.

Colorado Department of Transportation
902 Erie Avenue
Pueblo, CO 81001
Phone: 719-546-5438 FAX: 719-546-5702
Region 2 KSR

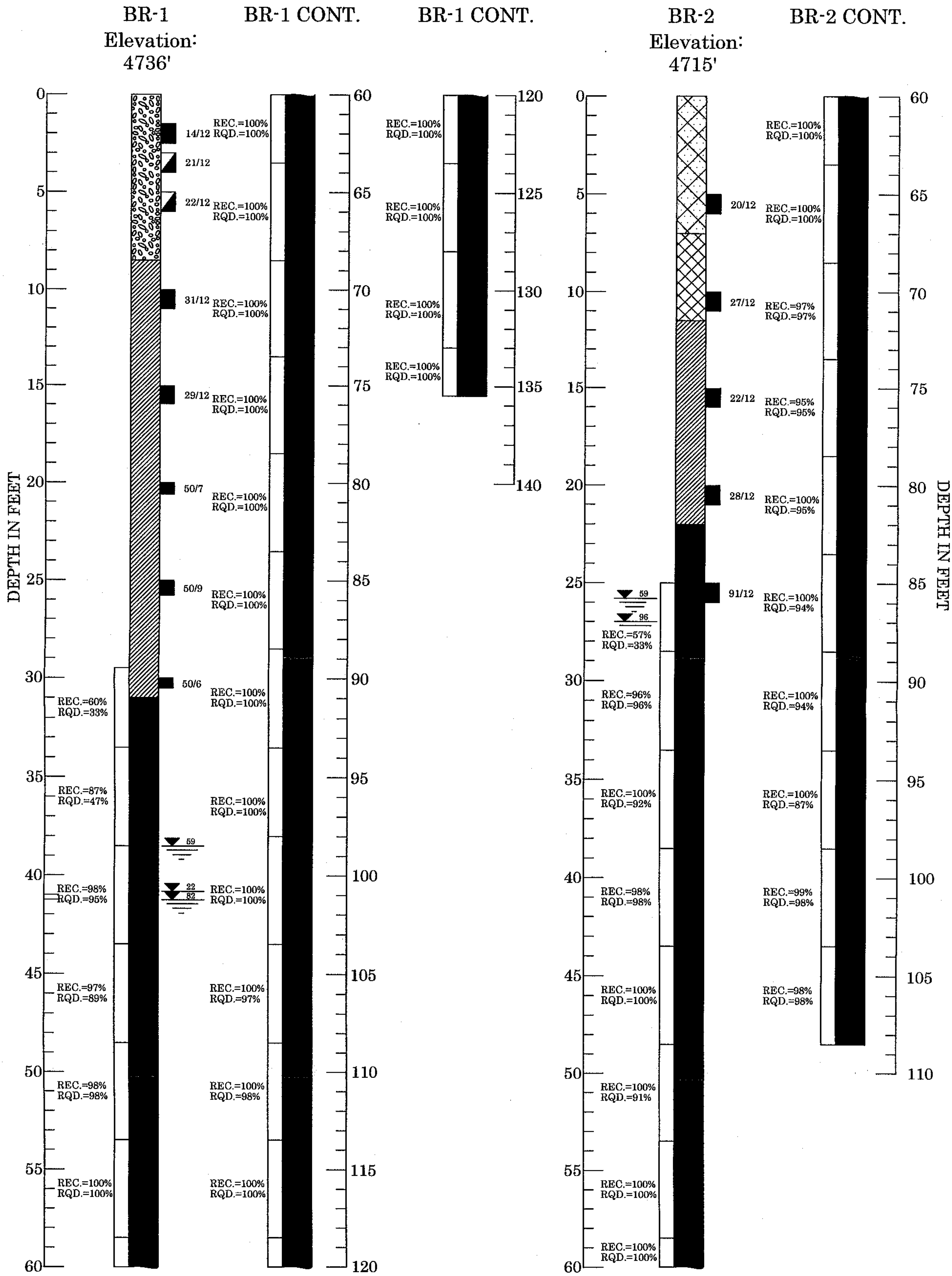
As Constructed
No Revisions:
Revised:
Void:

GEOTECHNICAL PLAN

Designer:	XXXXXXXX	Structure Numbers	K-18-GS (EB)
Detailer:	D. Anderson		K-18-GT (WB)
Sheet Subset:	G1 of G4	Subset Sheets:	XXX of XXX

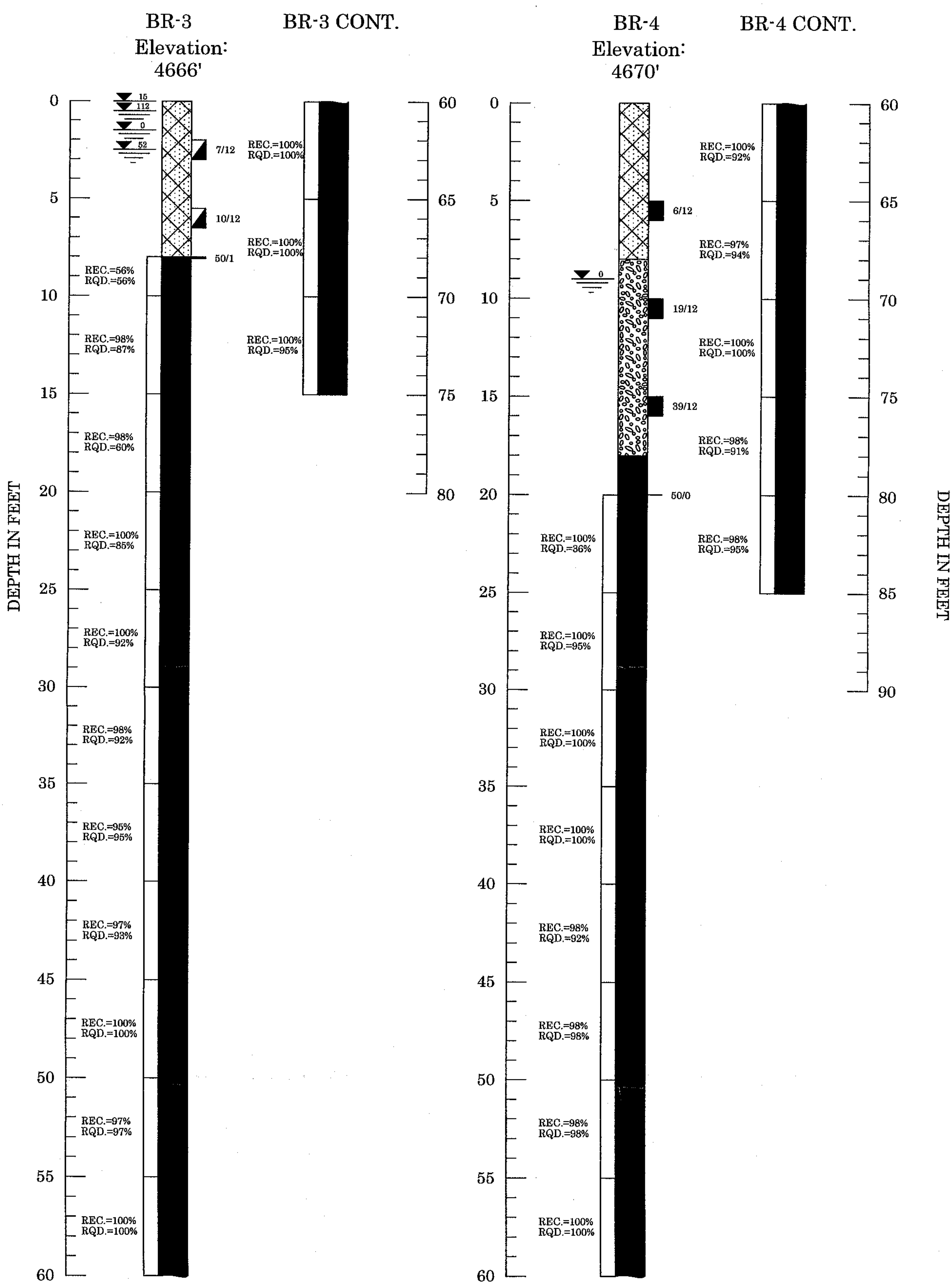
Project No./Code
BR 0961-008
13141
Sheet Number XXX

BORING LOGS



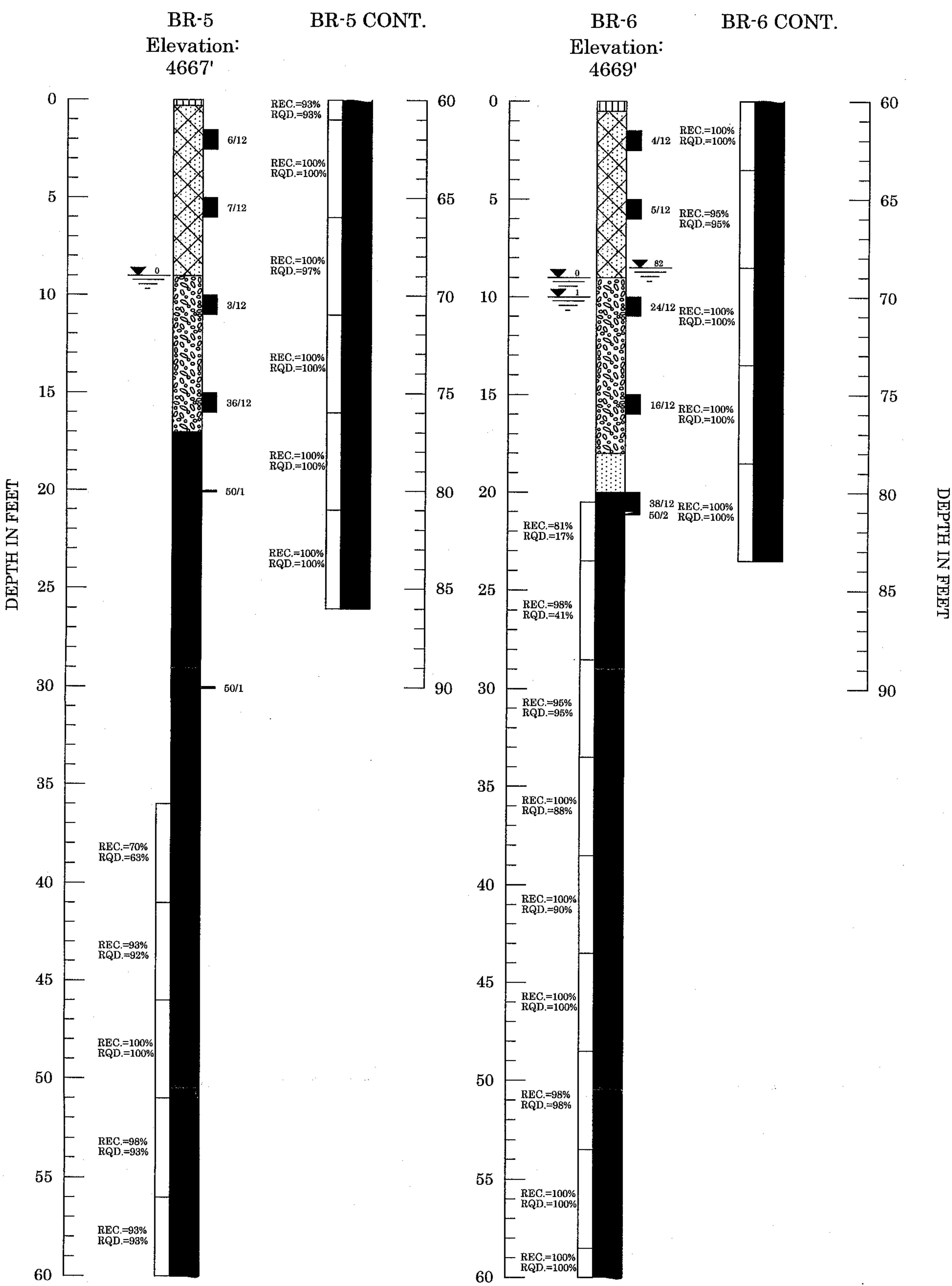
BORING LOGS	GOODSON & ASSOCIATES, INC. Consulting Engineers 12200 West 50th Place Wheat Ridge, Colorado 80033 (303) 233-2244	
SH 96A (4th Street) Bridge Over The Arkansas River Pueblo, Colorado	Project No.: 65569	Scale: 1" = 5'
	Date: October, 2006	Drawn by: DM
		Checked by: JF

BORING LOGS



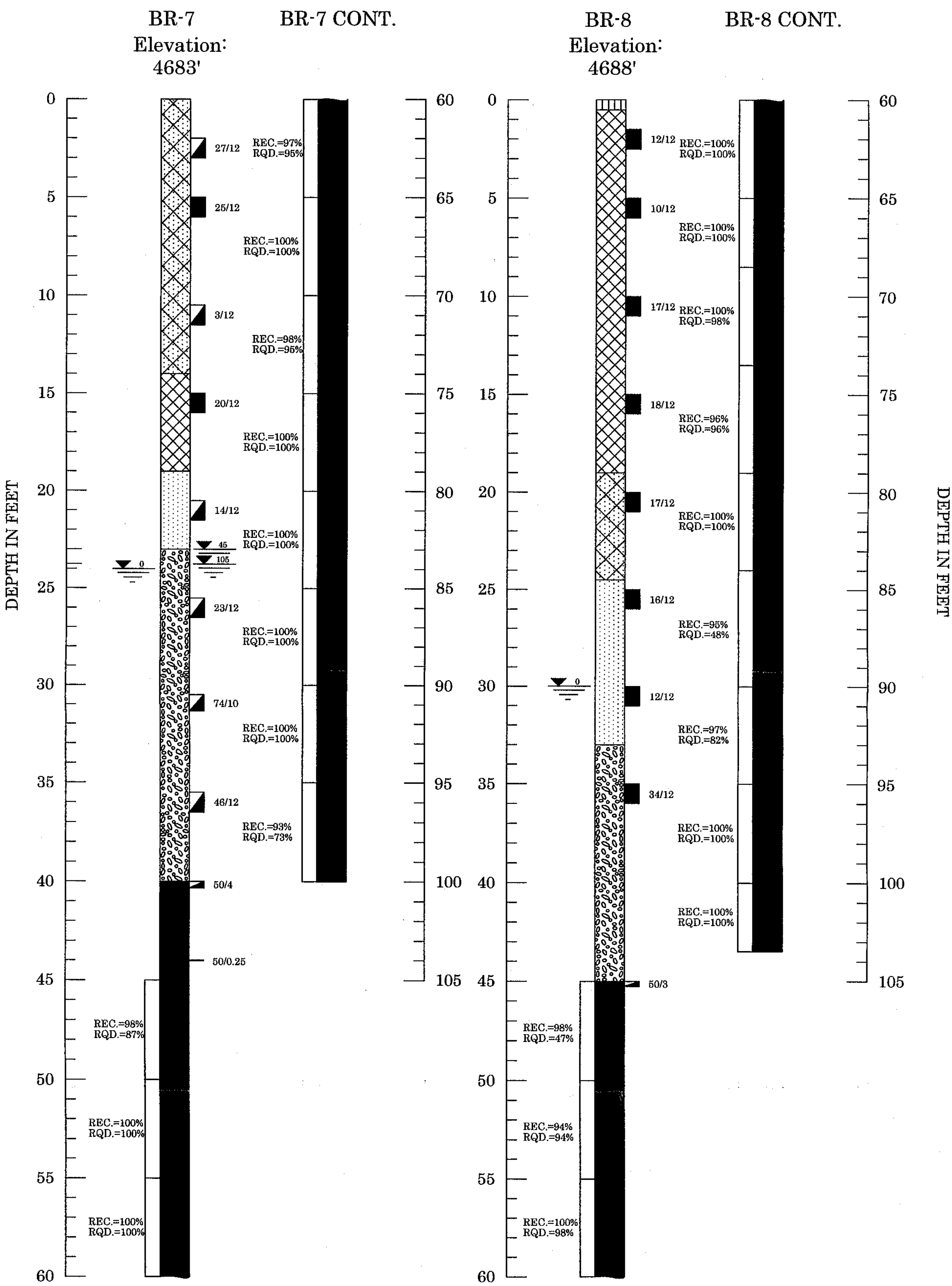
BORING LOGS	GAE GOODSON & ASSOCIATES, INC. Consulting Engineers 12200 West 50th Place Wheat Ridge, Colorado 80033 (303) 233-2244
SH 96A (4th Street) Bridge Over The Arkansas River Pueblo, Colorado	Project No.: 65569 / Scale: 1" = 5' / Date: October, 2006 / Drawn by: DM / Checked by: JF

BORING LOGS



BORING LOGS	GODSON & ASSOCIATES, INC. Consulting Engineers 12200 West 50th Place Wheat Ridge, Colorado 80033 (303) 233-2244	
SH 96A (4th Street) Bridge Over The Arkansas River Pueblo, Colorado	Project No.: 65569	Scale: 1" = 5'
	Date: October, 2006	Drawn by: DM
		Checked by: JF

BORING LOGS



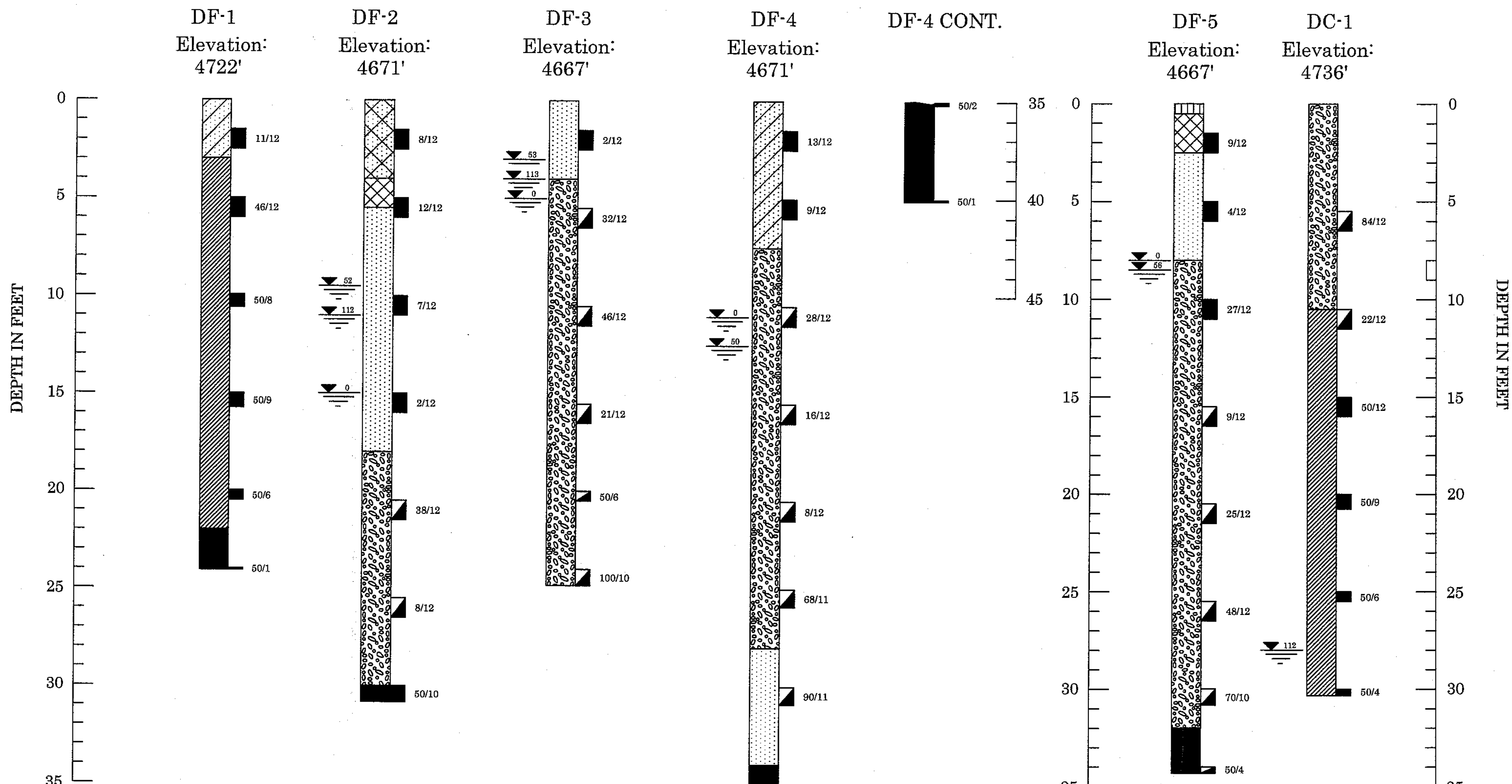
BORING LOGS

SH 96A (4th Street) Bridge
Over The Arkansas River
Pueblo, Colorado

GOODSON & ASSOCIATES, INC.
Consulting Engineers
12200 West 50th Place
Wheat Ridge, Colorado 80033
(303) 233-2244

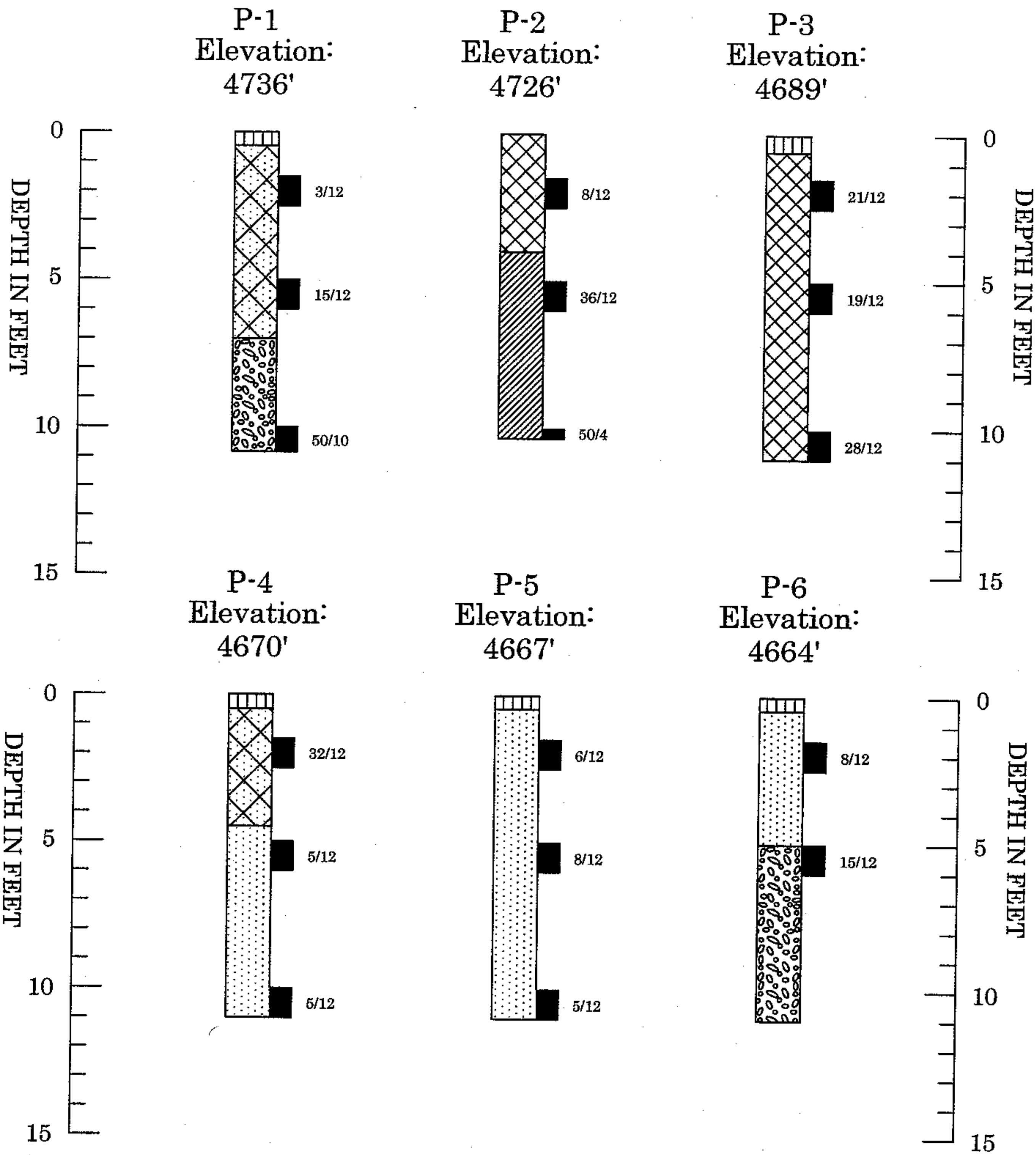
Project No.: 65569	Scale: 1" = 5'
Date: October, 2006	Drawn by: DM
	Checked by: JF

BORING LOGS



Project No.: 65569	Scale: 1" = 5'	BORING LOGS	SH 96A (4th Street) Bridge Over The Arkansas River Pueblo, Colorado	GOODSON & ASSOCIATES, INC. Consulting Engineers 12200 West 50th Place Wheat Ridge, Colorado 80033 (303) 233-2244
Date: October, 2006	Drawn By: DM			
	Checked By: JF			

BORING LOGS



Boring Logs	GOODSON & ASSOCIATES, INC. Consulting Engineers 12200 West 50th Place, Unit A Wheat Ridge, Colorado 80033 (303) 233-2244	
SH 96A (4th Street) Bridge Over The Arkansas River Pueblo, Colorado	Project No.: 65569	Scale: 1" = 5'
	Date: October, 2006	Drawn by: DM
		Checked by: JF

LEGEND FOR BORING LOGS

- PAVEMENT, asphalt thickness approximately 3 inches to 7 inches overlying aggregate base course of approximately 0 to 8 inches.

- FILL, SAND, clayey, silty, slightly gravelly to gravelly, fine to coarse grained, very loose to dense, slightly moist to wet, light brown, brown, dark brown, gray, occasional construction debris, rock fragments, wood fragments, cobbles, and iron staining.

- FILL, CLAY, sandy, stiff to very stiff, slightly moist to very moist, with occasional claystone fragments, light brown to dark brown, dark gray to black, occasionally calcareous, occasional sandy silt.

- SAND, fine to coarse grained, silty, very loose to very dense, occasional gravel, slightly moist to wet, tan to light brown.

- CLAY, sandy, silty, soft to stiff, moist, dark gray.

- SAND, fine to coarse grained, gravelly, very loose to very dense, slightly moist to wet, light brown, gray, occasionally sandy gravel.

- CLAYSTONE, slightly sandy to sandy, firm to very hard, slightly moist to moist, orange-brown, light brown to dark brown, occasional iron staining, occasionally calcareous.

- SHALE, calcareous (chalky), occasionally slightly sandy, very hard, slightly weathered to unweathered, strong to very strong, slightly moist to moist, dark gray to light gray, fractures typically are joints, very narrow to wide, no infilling to clay/claystone infilling, planar and irregular, smooth to rough, approximately 0° to 45°, wide to moderately wide spacing.

Legend for Boring Logs	GOODSON & ASSOCIATES, INC. Consulting Engineers 12200 West 50th Place, Unit A Wheat Ridge, Colorado 80033 (303) 233-2244	
SH 96A (4th Street) Bridge Over The Arkansas River Pueblo, Colorado	Project No.: 65569	Scale: NONE
	Date: October, 2006	Drawn by: DM
		Checked by: JF

LEGEND FOR BORING LOGS CONTINUED

13/12 – Drive Sample. Indicates thirteen blows of a one hundred forty pound hammer falling thirty inches were required to drive the two inch I.D. California Barrel sampler twelve inches.

13/12 – Drive Sample. Indicates thirteen blows of a one hundred forty pound hammer falling thirty inches were required to drive the two inch O.D. Split Spoon sampler twelve inches.

REC.=93%
RQD.=57% – Indicates core sample obtained.
REC= Percent Recovery
RQD= Rock Quality Designation

3 – Indicates groundwater level and number of days after drilling measurement was made.

Legend for Boring Logs	GOODSON & ASSOCIATES, INC. Consulting Engineers 12200 West 50th Place, Unit A Wheat Ridge, Colorado 80033 (303) 233-2244	
SH 96A (4th Street) Bridge Over The Arkansas River Pueblo, Colorado	Project No.: 65569	Scale: NONE
	Date: October, 2006	Drawn by: DM
		Checked by: JF

NOTES FOR BORING LOGS

(1) Exploratory borings were drilled between March 27 and August 17, 2006, with 4 1/4-inch I.D. continuous flight hollow stem power auger, 4-inch solid stem power auger, and HX rock coring with 2.4-inch I.D. HQ drill bit.

(2) Locations of the exploratory borings were determined by the project surveyors after the drilling activities were completed.

(3) Elevations of the exploratory borings were provided by the project surveyors. The logs of the exploratory borings are drawn to depth.

(4) The exploratory boring locations should be considered accurate only to the degree implied by the method used.

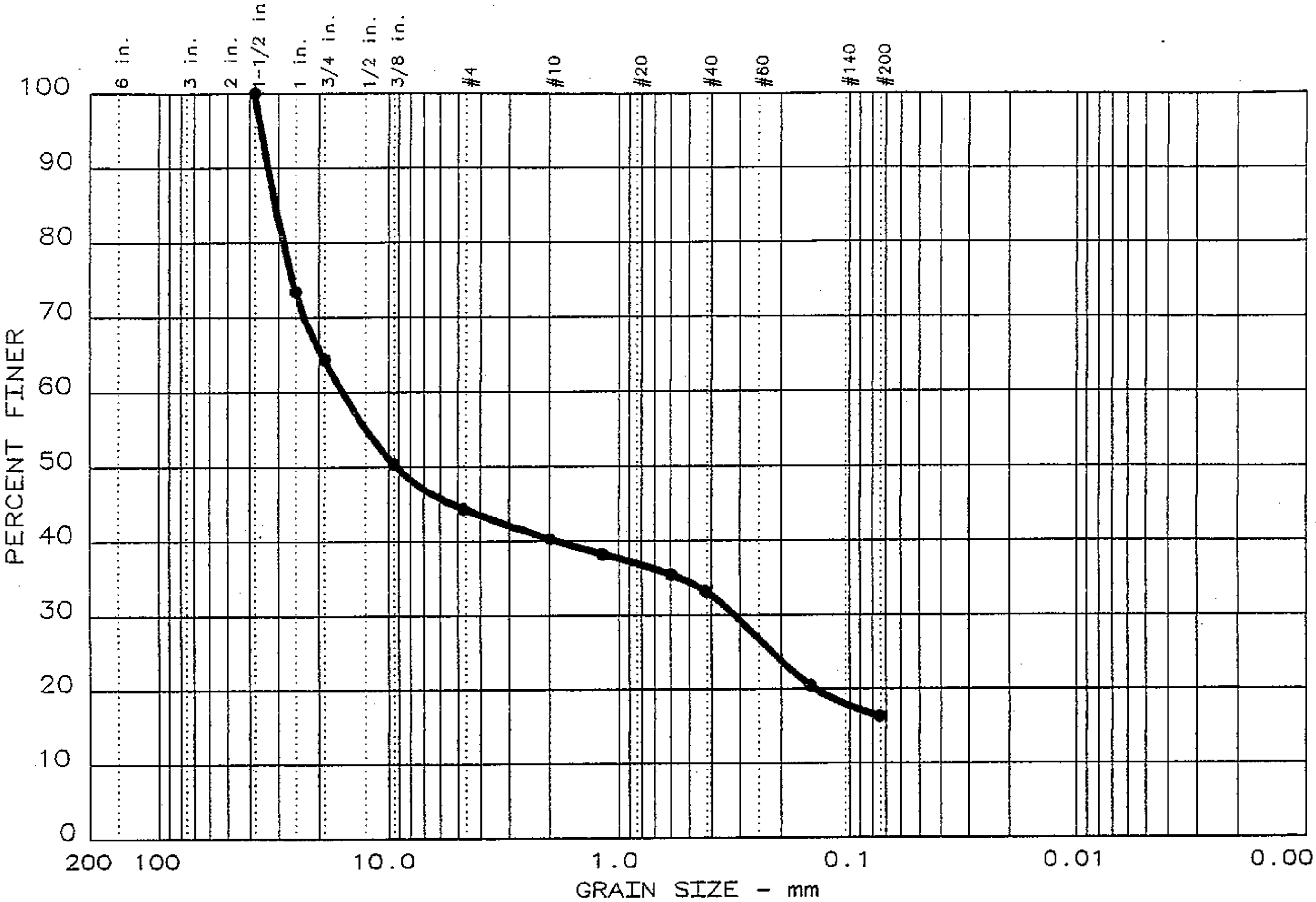
(5) The boundary lines between materials shown on the exploratory boring logs are approximate and the transitions between strata may be gradual.

(6) Ground water levels shown on the logs were measured at the time and under conditions indicated. Fluctuations in the groundwater levels may occur with time.

Notes for Boring Logs	GOODSON & ASSOCIATES, INC. Consulting Engineers 12200 West 50th Place, Unit A Wheat Ridge, Colorado 80033 (303) 233-2244	
SH 96A (4th Street) Bridge Over The Arkansas River Pueblo, Colorado	Project No.: 65569	Scale: NONE
	Date: October, 2006	Drawn by: DM
		Checked by: JF

APPENDIX B

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	55.7	27.9	16.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	31.0	15.7	9.25	0.317				

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Gravel with Sand	GM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-1 @ 0'-5'

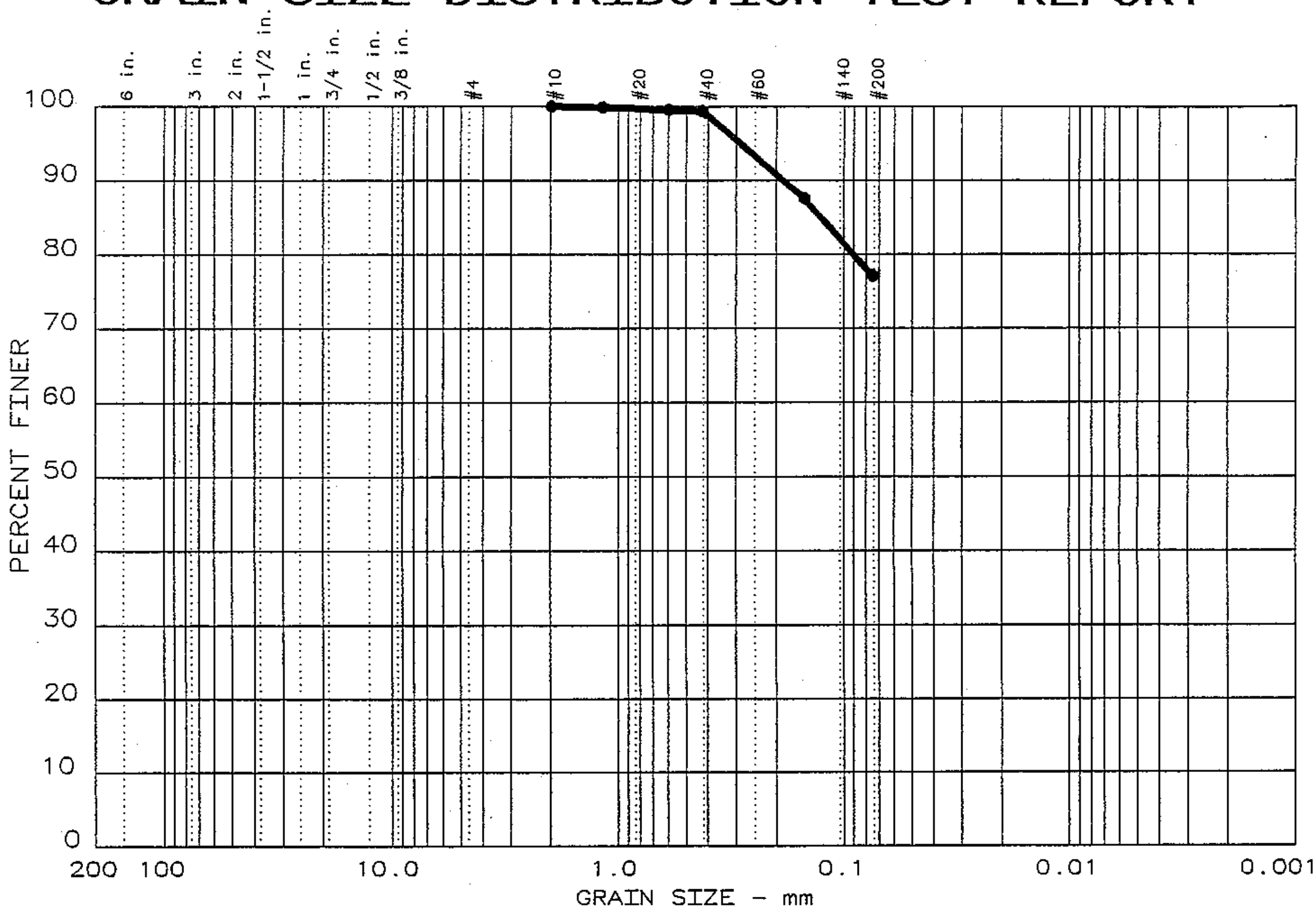
Date: 06.20.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	22.9	77.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	52	25	0.126							

MATERIAL DESCRIPTION	USCS	AASHTO
● Fat Clay with Sand	CH	A-7-6(20)

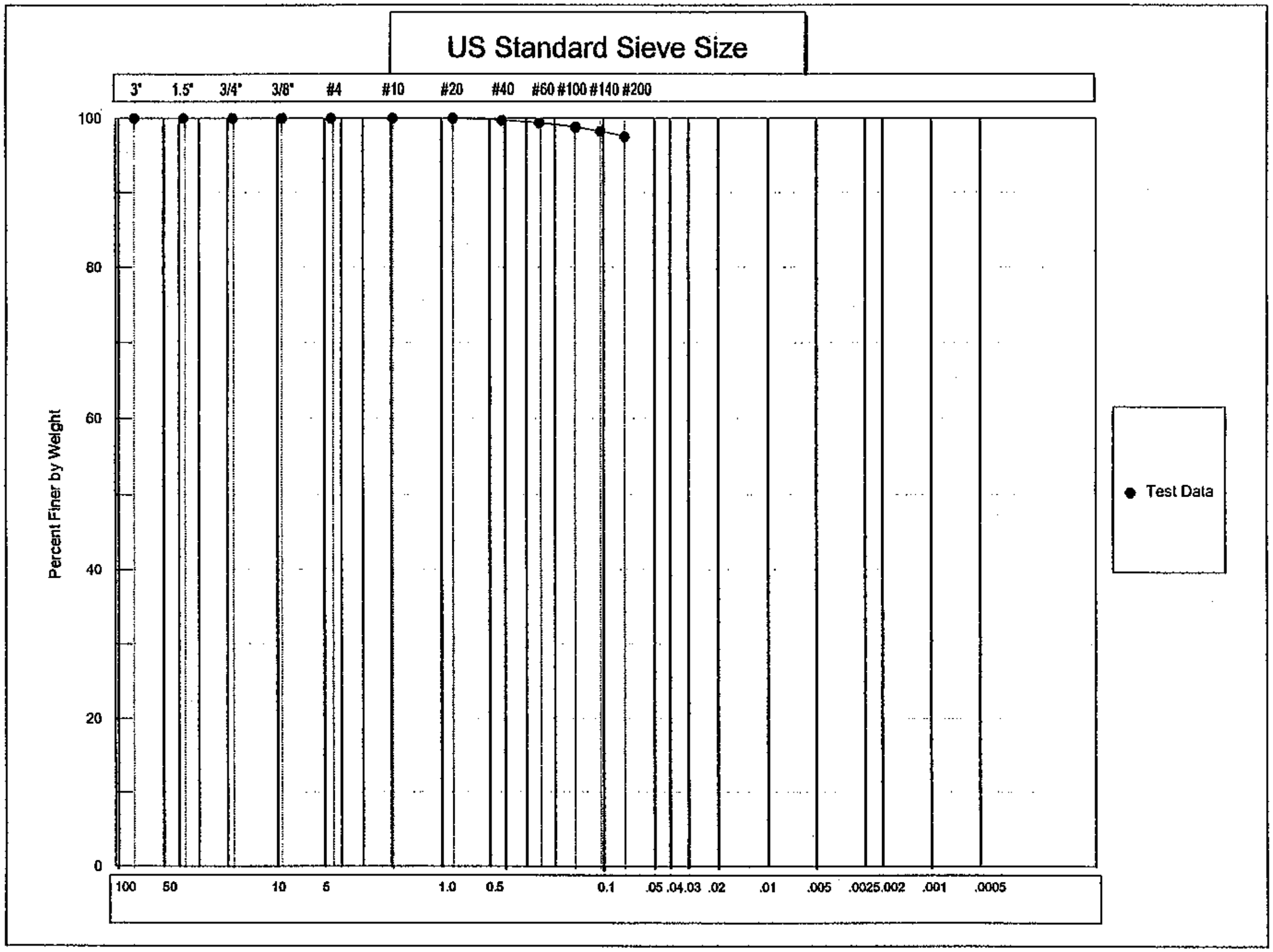
Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-1 @ 10'

Date: 08.03.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______

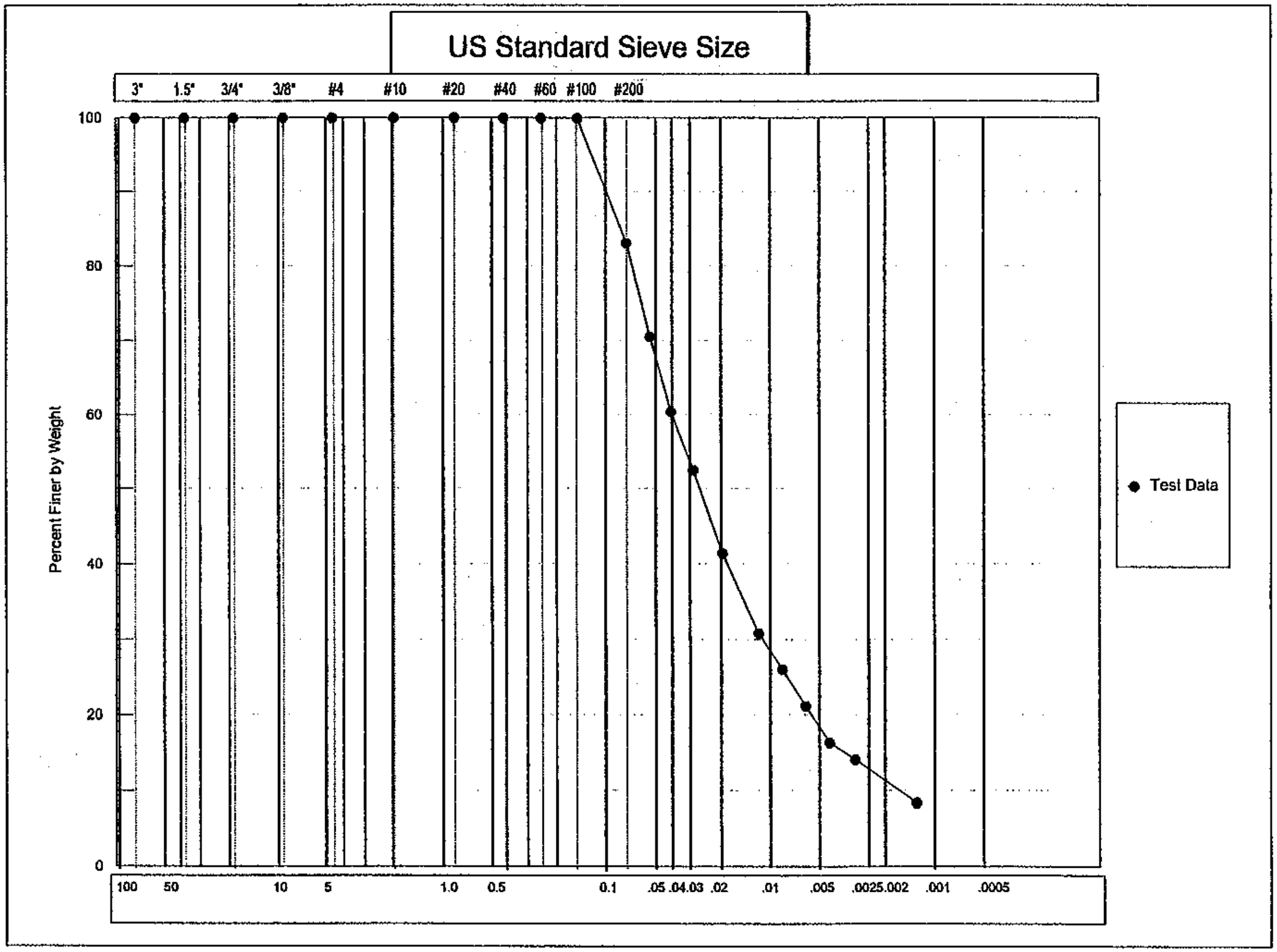


COBBLES	GRAVEL		SAND			SILT OR CLAY	USCS
	COARSE	FINE	CRS	MEDIUM	FINE		

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	WENTWORTH
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			

Client: Goodson & Associates Boring No.: BR-1 Sample No.:
Job Number: 2014-104 Depth: 15'
Classification: **Classification Not Performed**

Advanced Terra Testing, Inc.

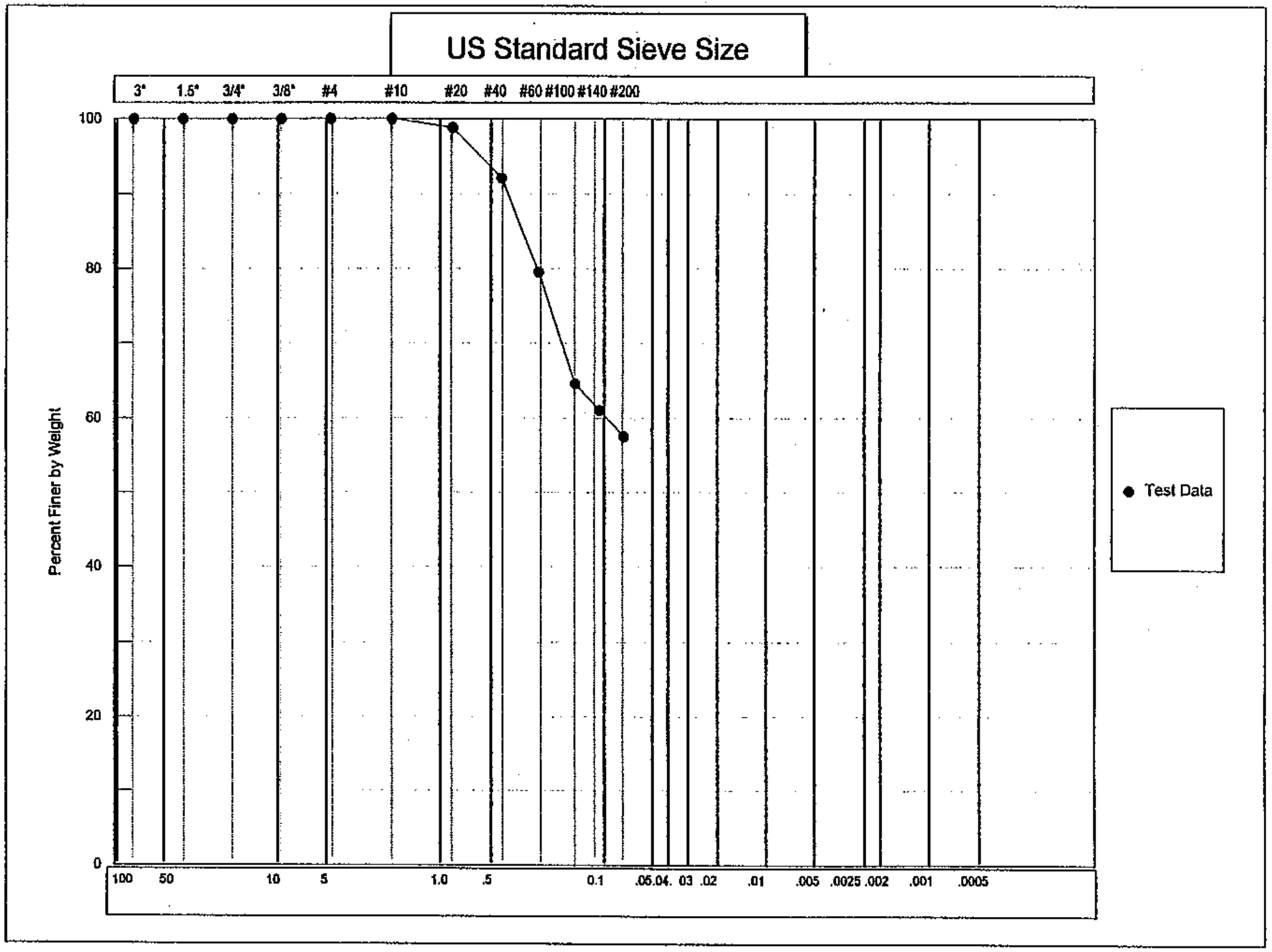


COBBLES	GRAVEL		SAND			SILT OR CLAY	USCS
	COARSE	FINE	CRS	MEDIUM	FINE		

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	WENTWORTH
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			

Client: Goodson & Associates Boring No.: BR-1 Sample No.:
Job Number: 2014-104 Depth: 42.0-42.2
Classification: **Classification Not Performed**

Advanced Terra Testing, Inc.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	CRS	MEDIUM	FINE	

USCS

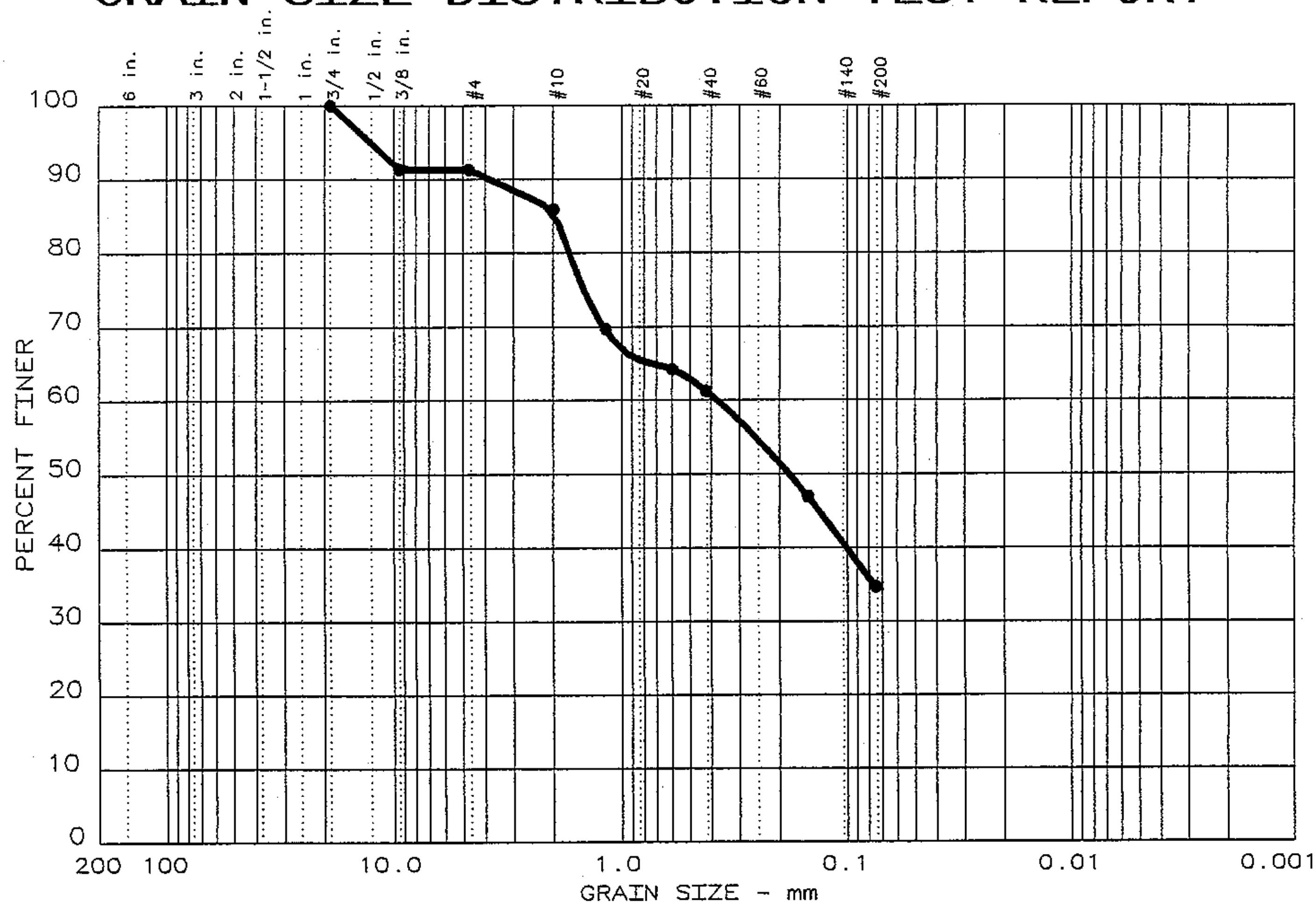
COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE		

WENTWORTH

Client: Goodson & Associates Boring No.: BR-1 Sample No.: Rock Core
Job Number: 2014-104 Depth: 47.0-48.5
Classification____________

Advanced Terra Testing, Inc.

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	8.7	56.6	34.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	1.95	0.376	0.182					

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Sand	SM	A-2-4(0)

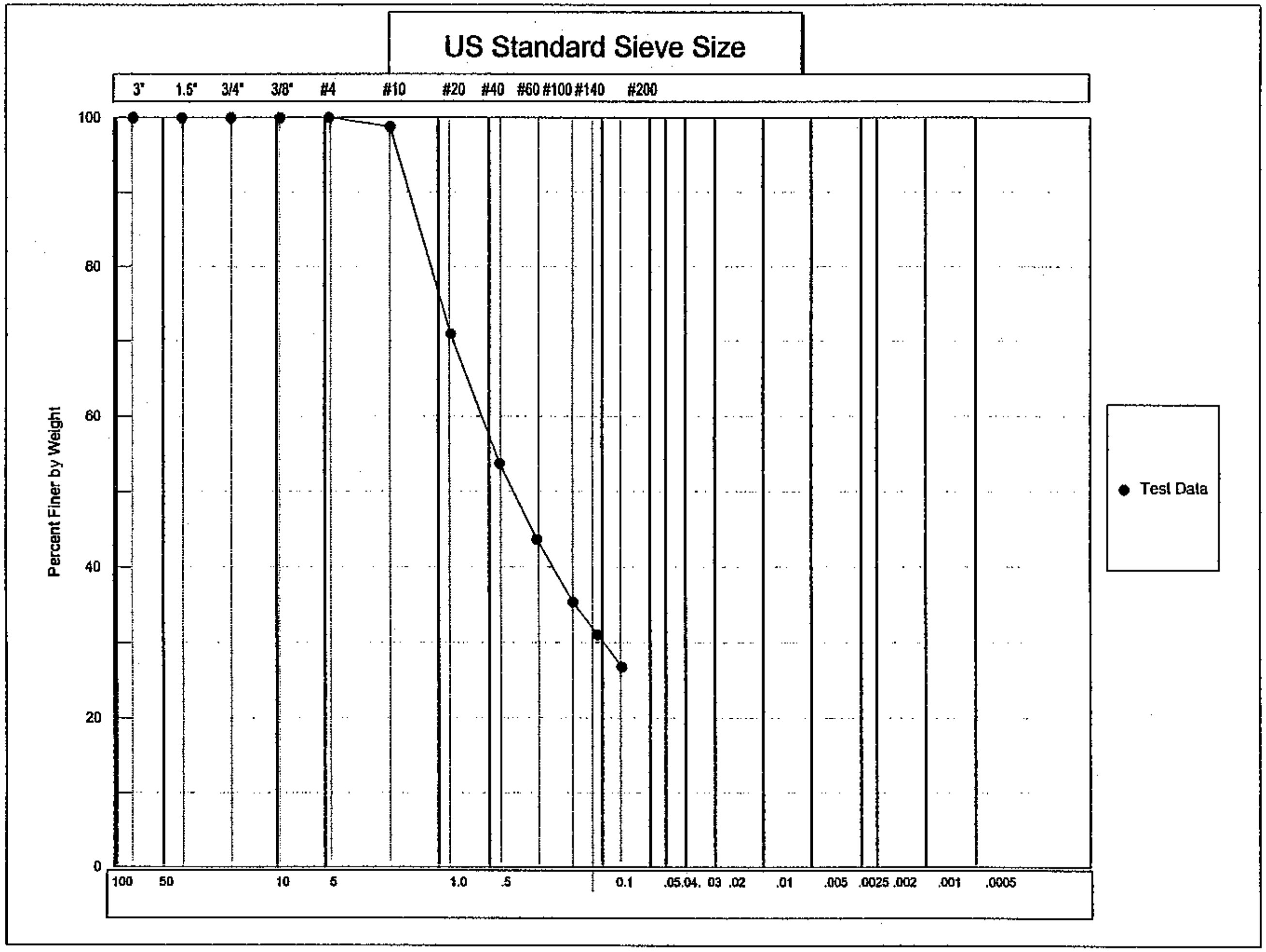
Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-2 @ 5'

Date: 08.03.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	CRS	MEDIUM	FINE	

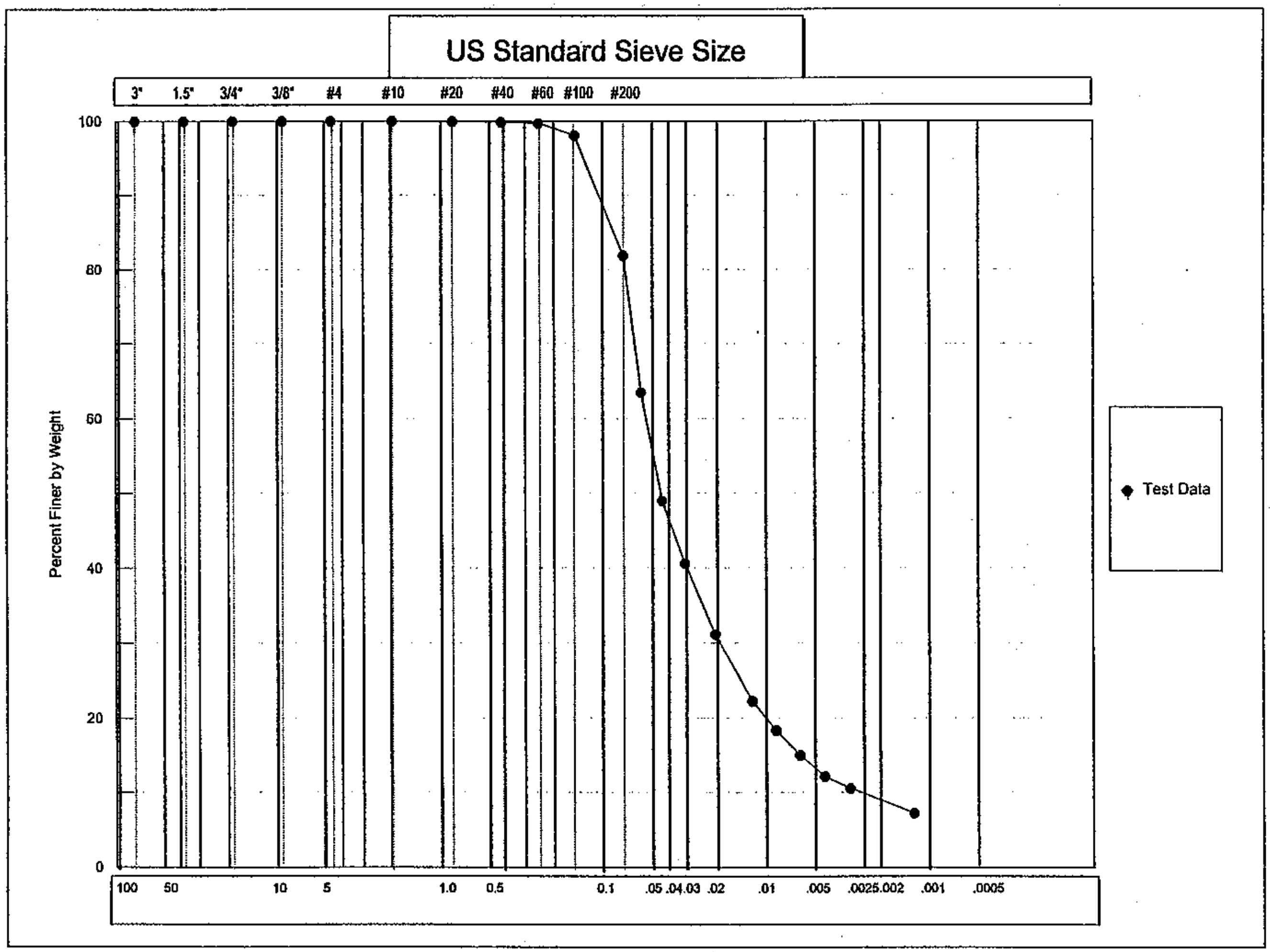
USCS

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE		

WENTWORTH

Client: Goodson & Associates Boring No.: BR-2 Sample No.: Rock Core
Job Number: 2014-104 Depth: 38.5-39.5'
Classification________________

Advanced Terra Testing, Inc.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	CRS	MEDIUM	FINE	

USCS

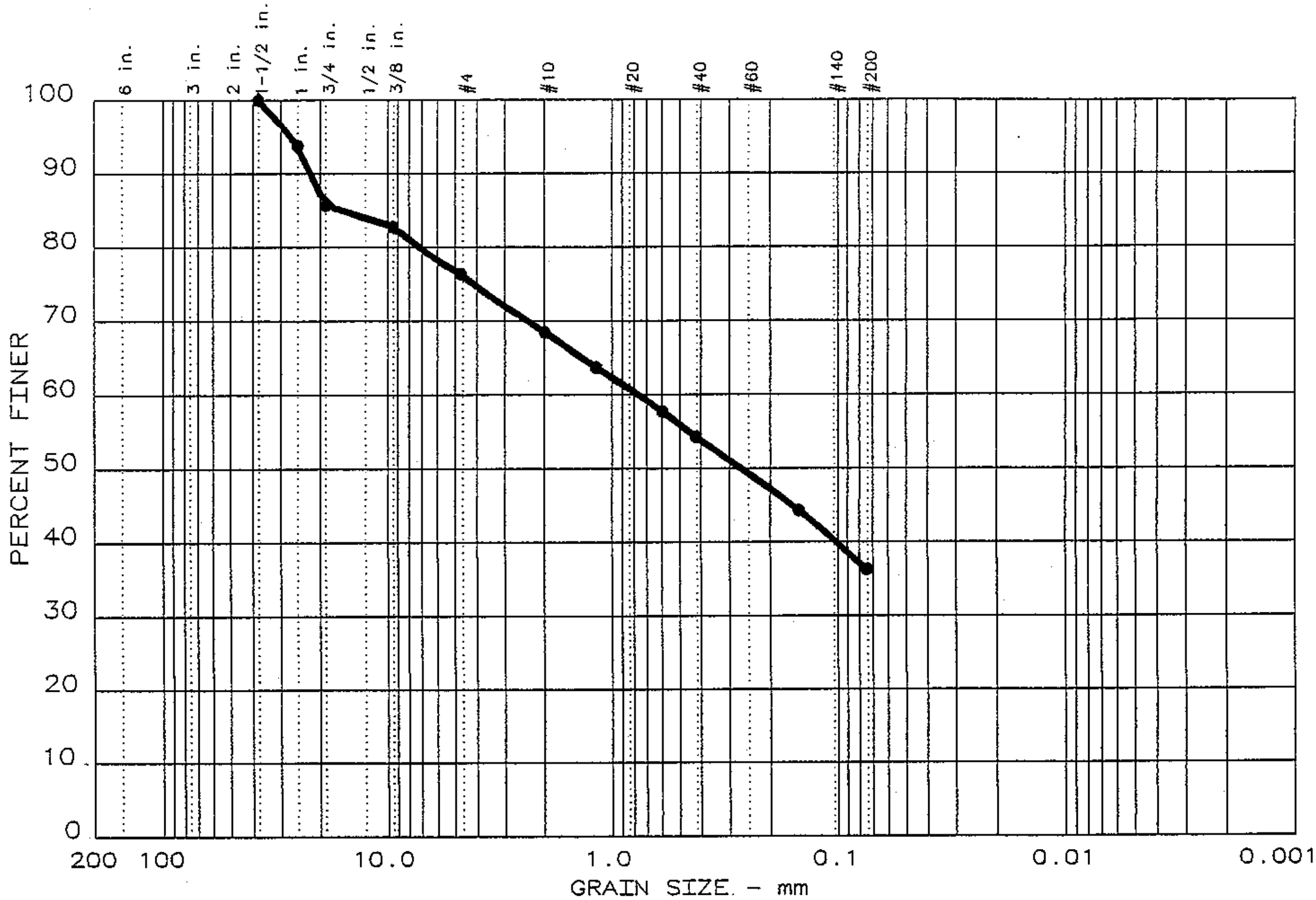
COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE		

WENTWORTH

Client: Goodson & associates Boring No.: BR-2 Sample No.: 2" Bentonite Seam
Job Number: 2014-104 Depth: 75.5-76.2
Classification: **Classification Not Performed**

Advanced Terra Testing, Inc.

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	23.6	40.1	36.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	32	18	16.2	0.767	0.266					

MATERIAL DESCRIPTION	USCS	AASHTO
● Clayey Sand with Gravel	SC	A-6(2)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-3 @ 5'

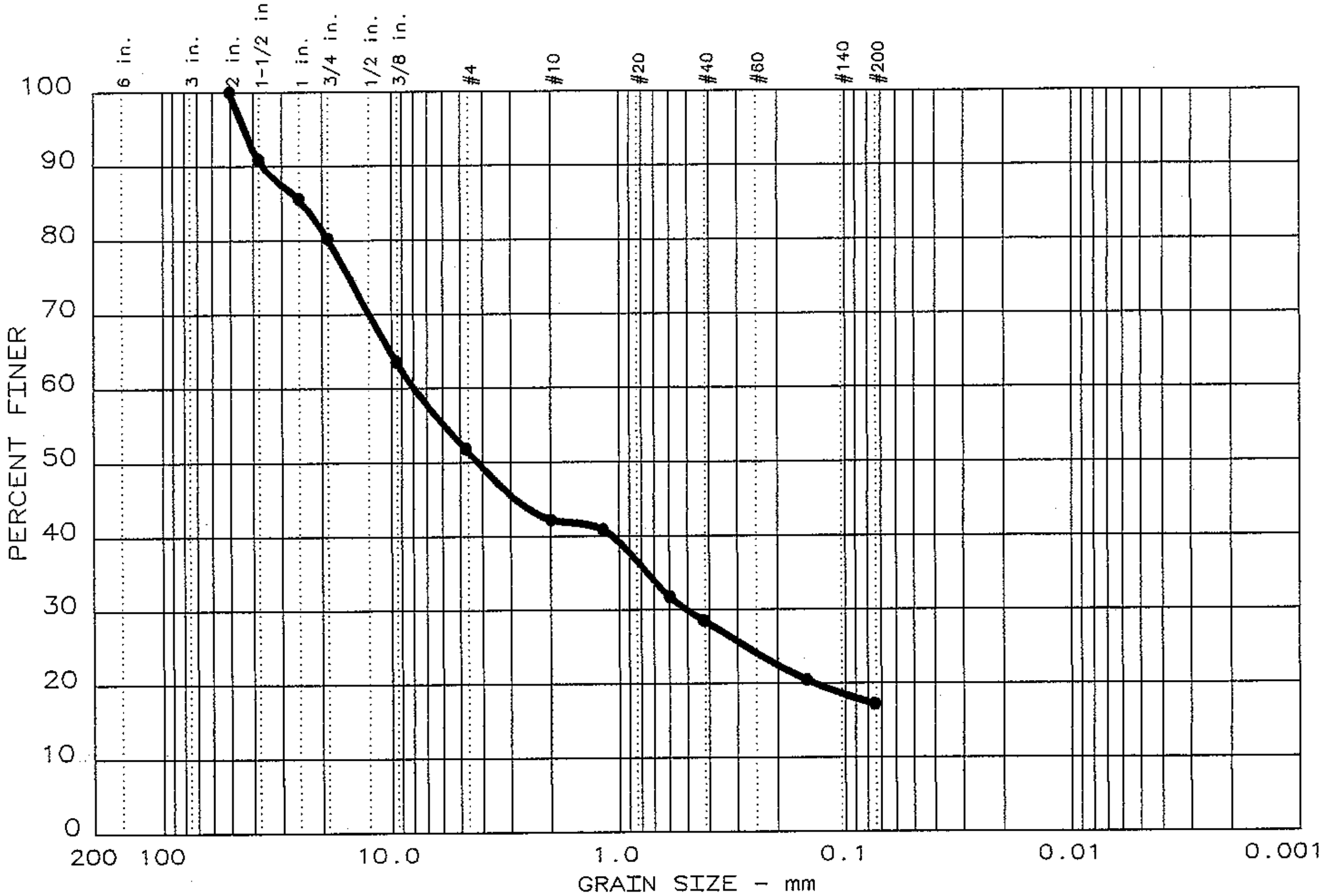
Date: 08.03.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	48.2	34.6	17.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	28	5	24.3	7.94	4.17	0.507				

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Gravel with Sand	GM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-4 @ 0'-5'

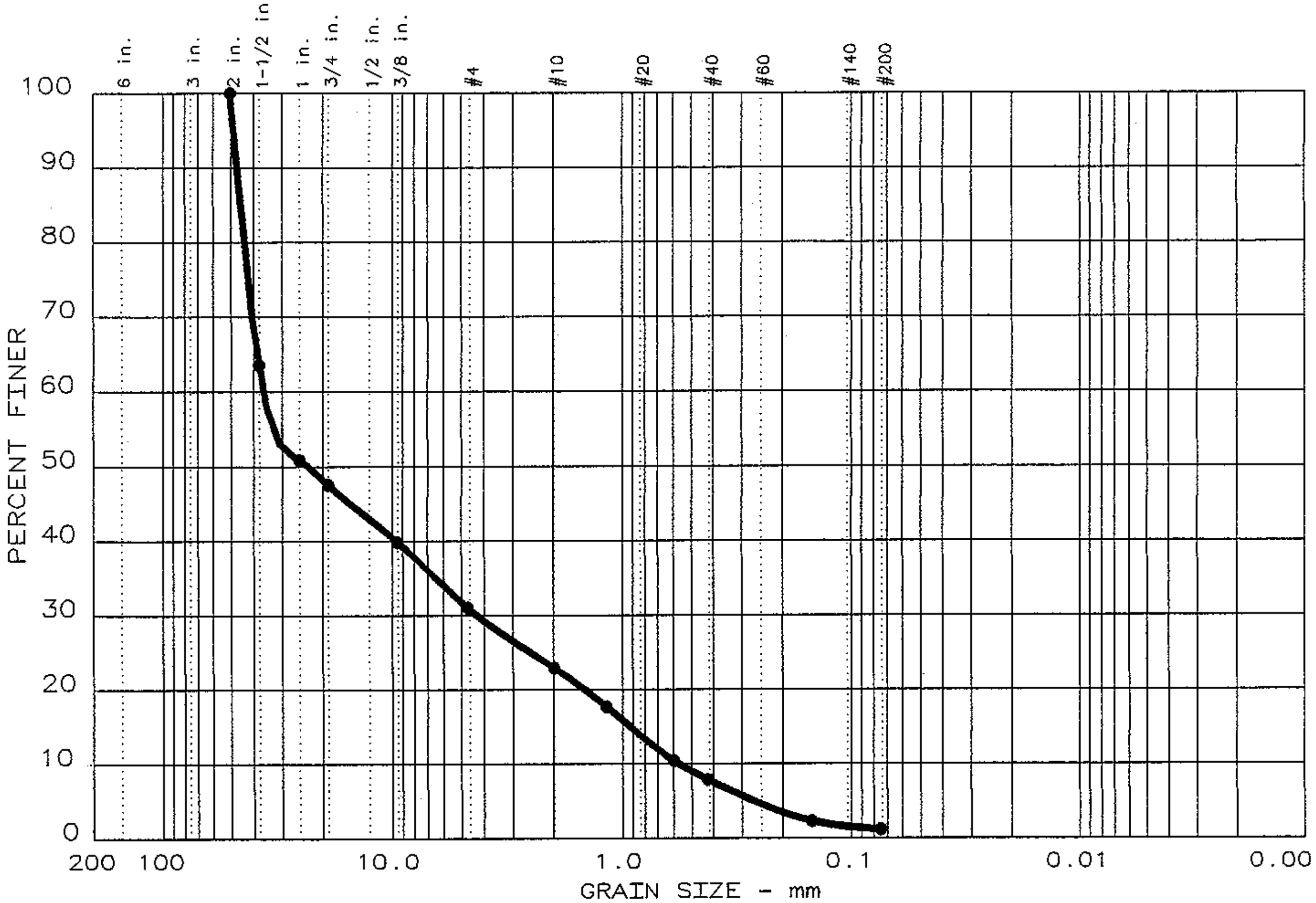
Date: 09.08.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	68.9	29.9	1.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	45.8	36.3	23.2	4.27	0.924	0.570	0.88	63.8

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Gravel with Sand	GP	A-1-a

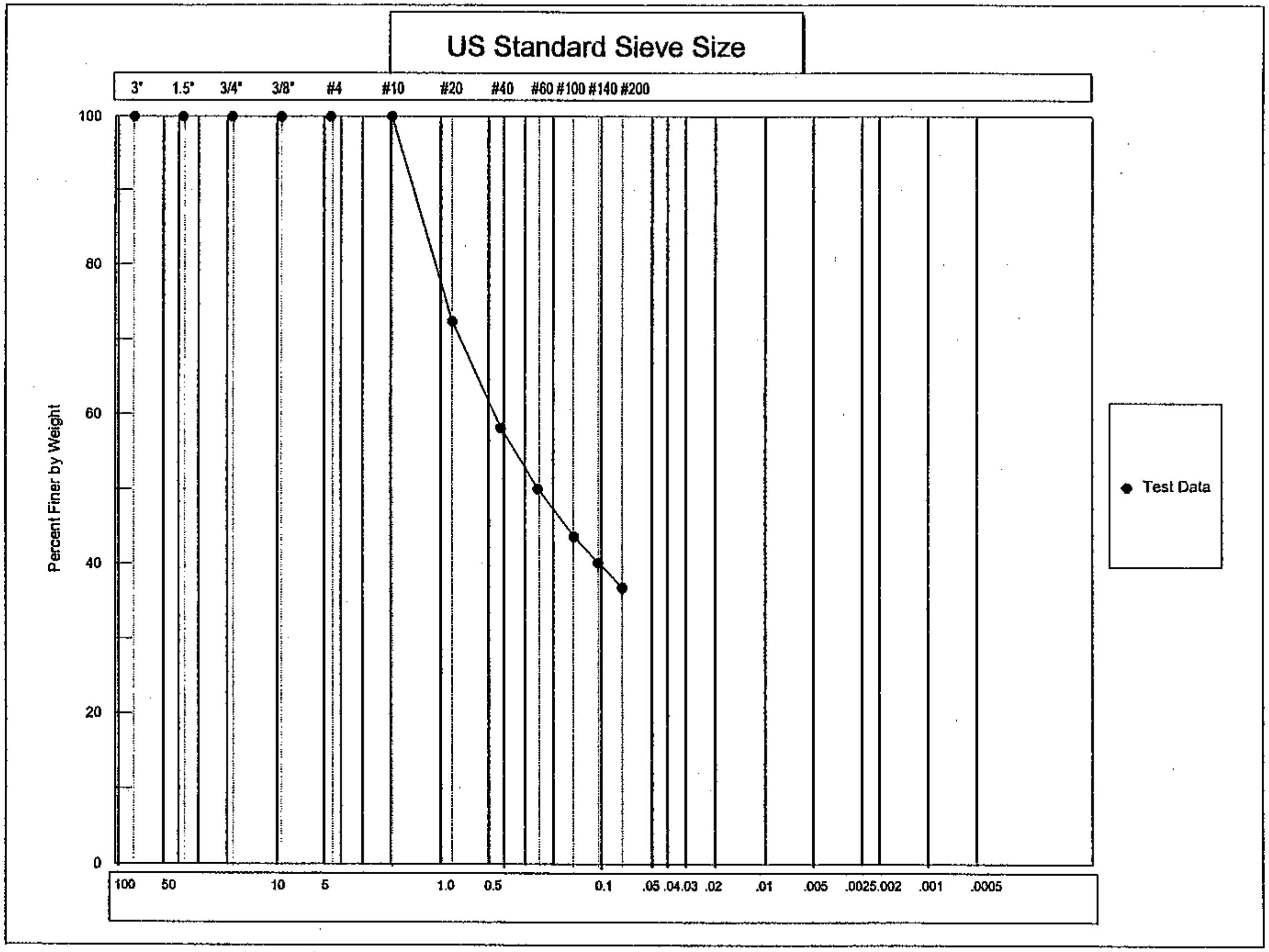
Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-4 @ 10'

Date: 09.08.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______



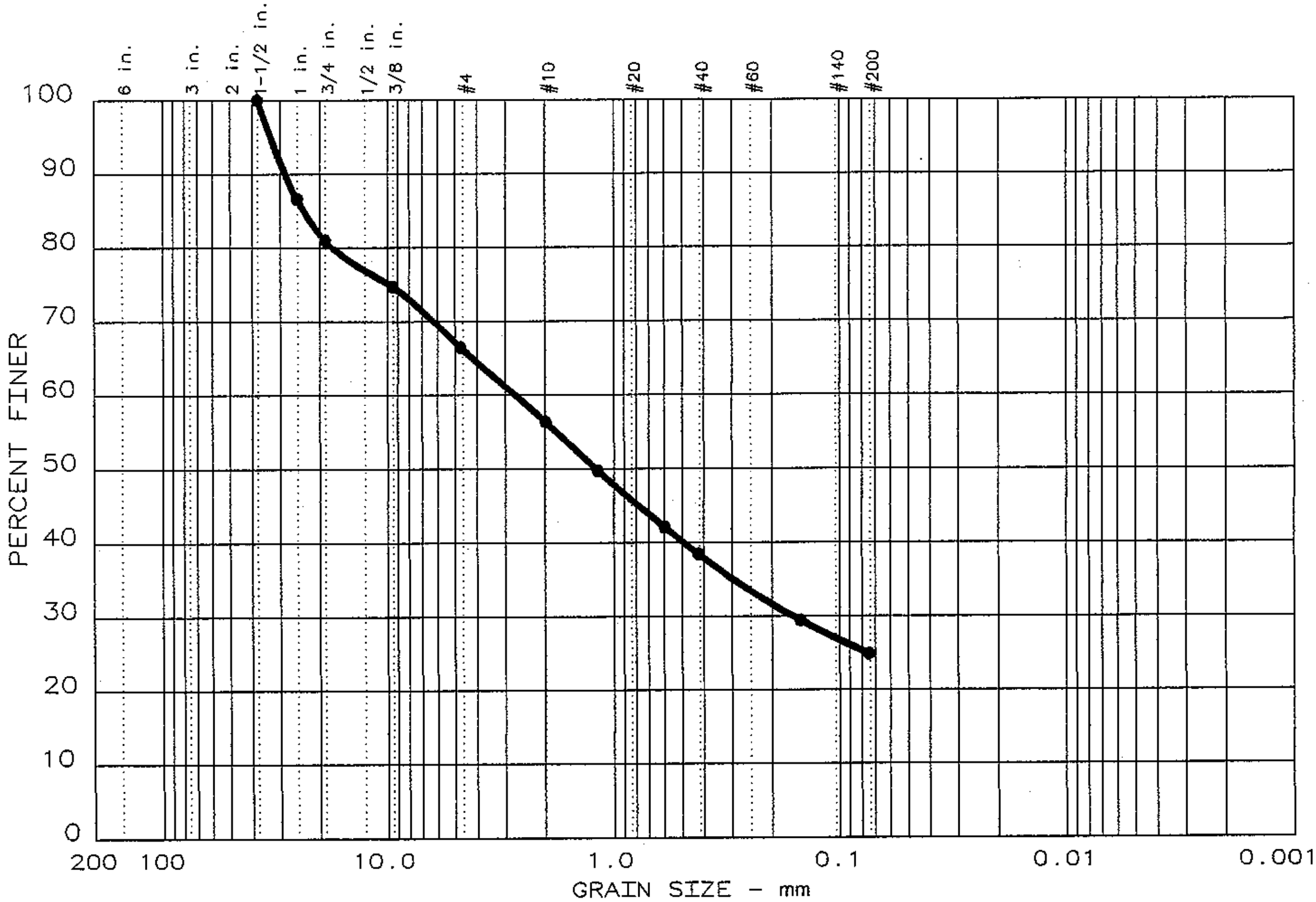
COBBLES	GRAVEL		SAND			SILT OR CLAY	USCS
	COARSE	FINE	CRS	MEDIUM	FINE		

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	WENTWORTH
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			

Client: Goodson & Associates Boring No.: BR-4 Sample No.:
Job Number: 2014-106 Depth: 46.8-48.2
Classification: **Classification Not Performed**

Advanced Terra Testing, Inc.

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	33.5	41.6	24.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	35	4	23.7	2.72	1.20	0.160				

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Sand with Gravel	SM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-5 @ 1'-5'

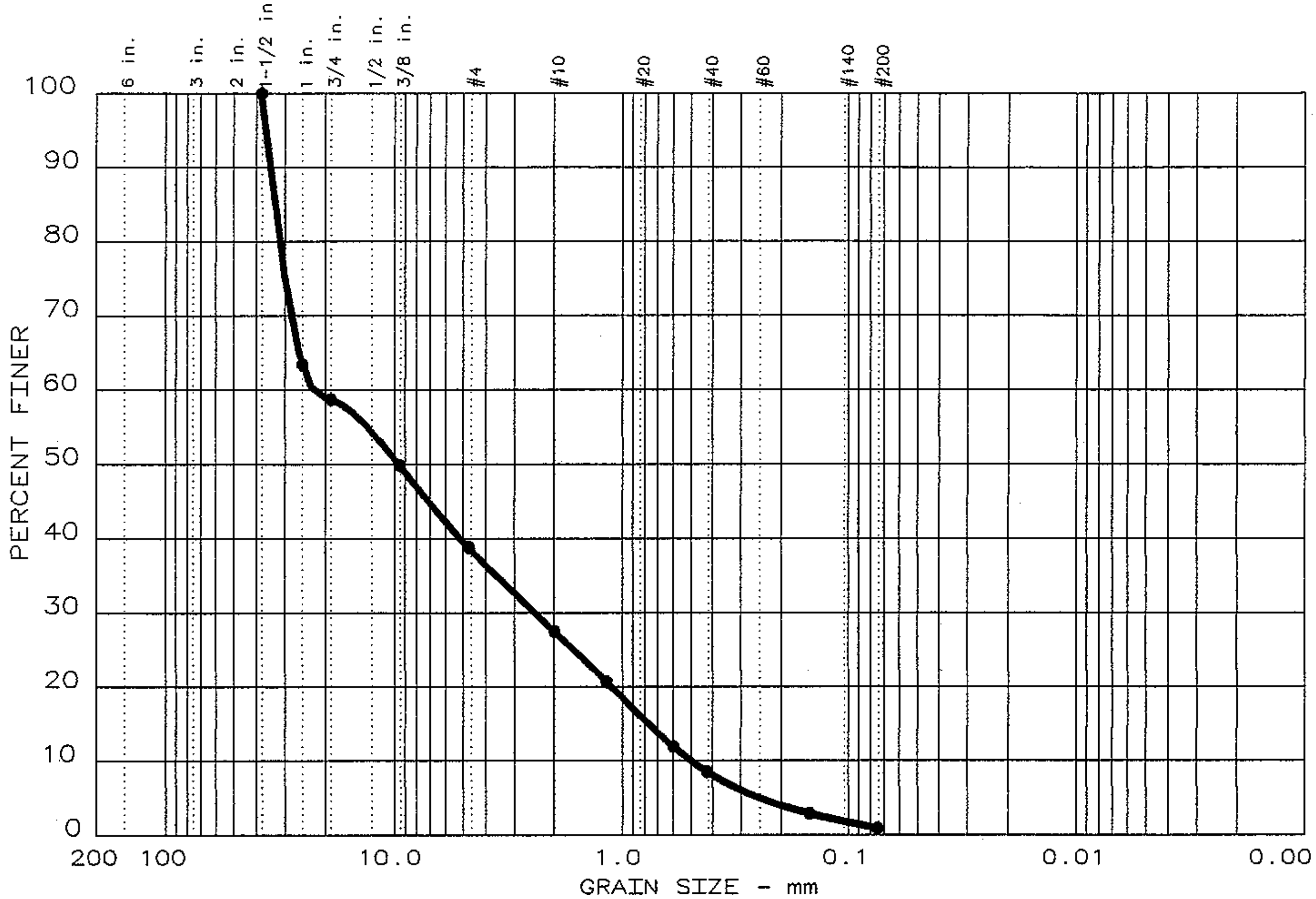
Date: 09.08.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	61.2	37.9	0.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	33.2	22.5	9.58	2.44	0.770	0.497	0.53	45.2

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Gravel with Sand	GP	A-1-a

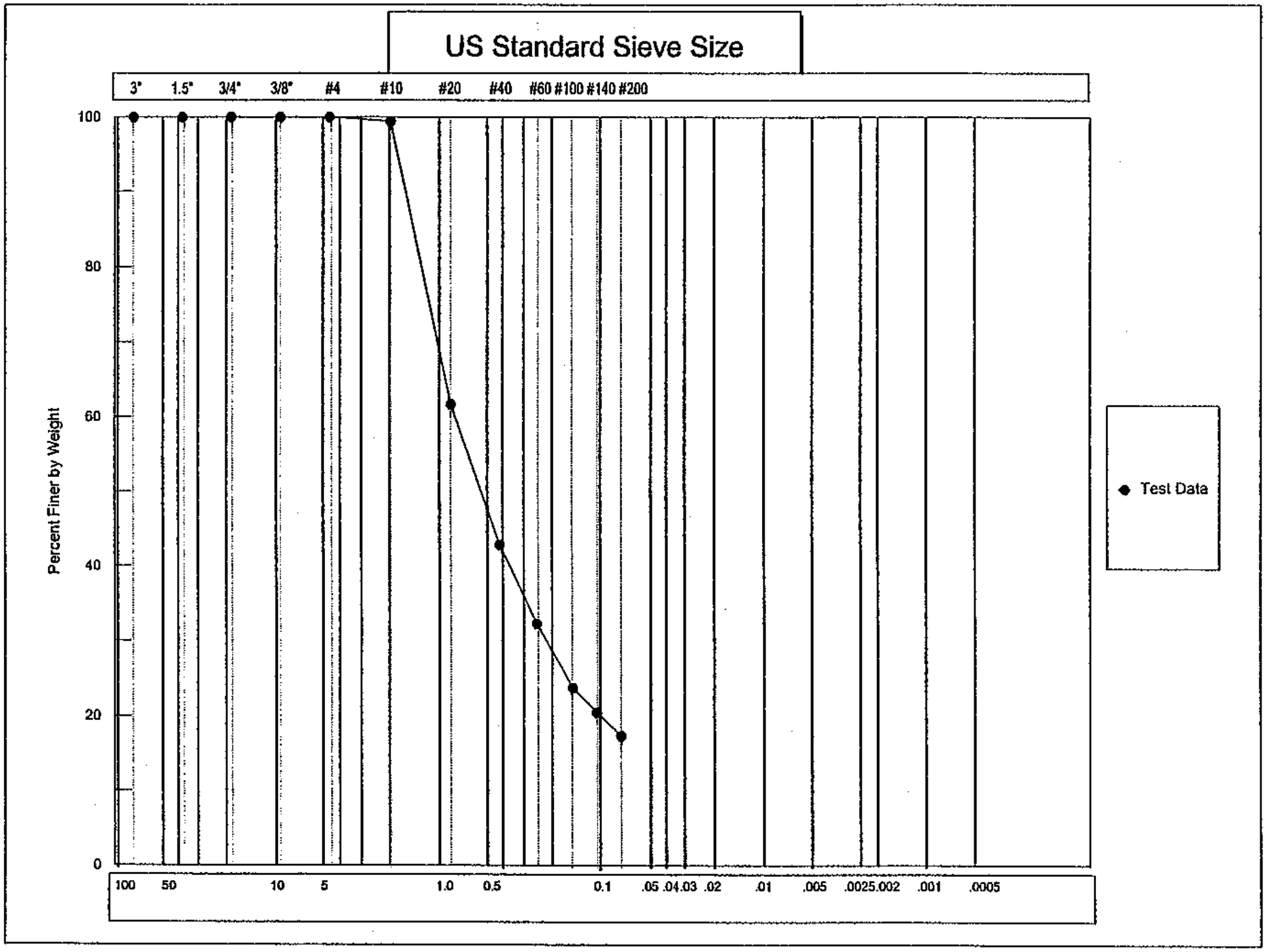
Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-5 @ 10'

Date: 09.08.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______



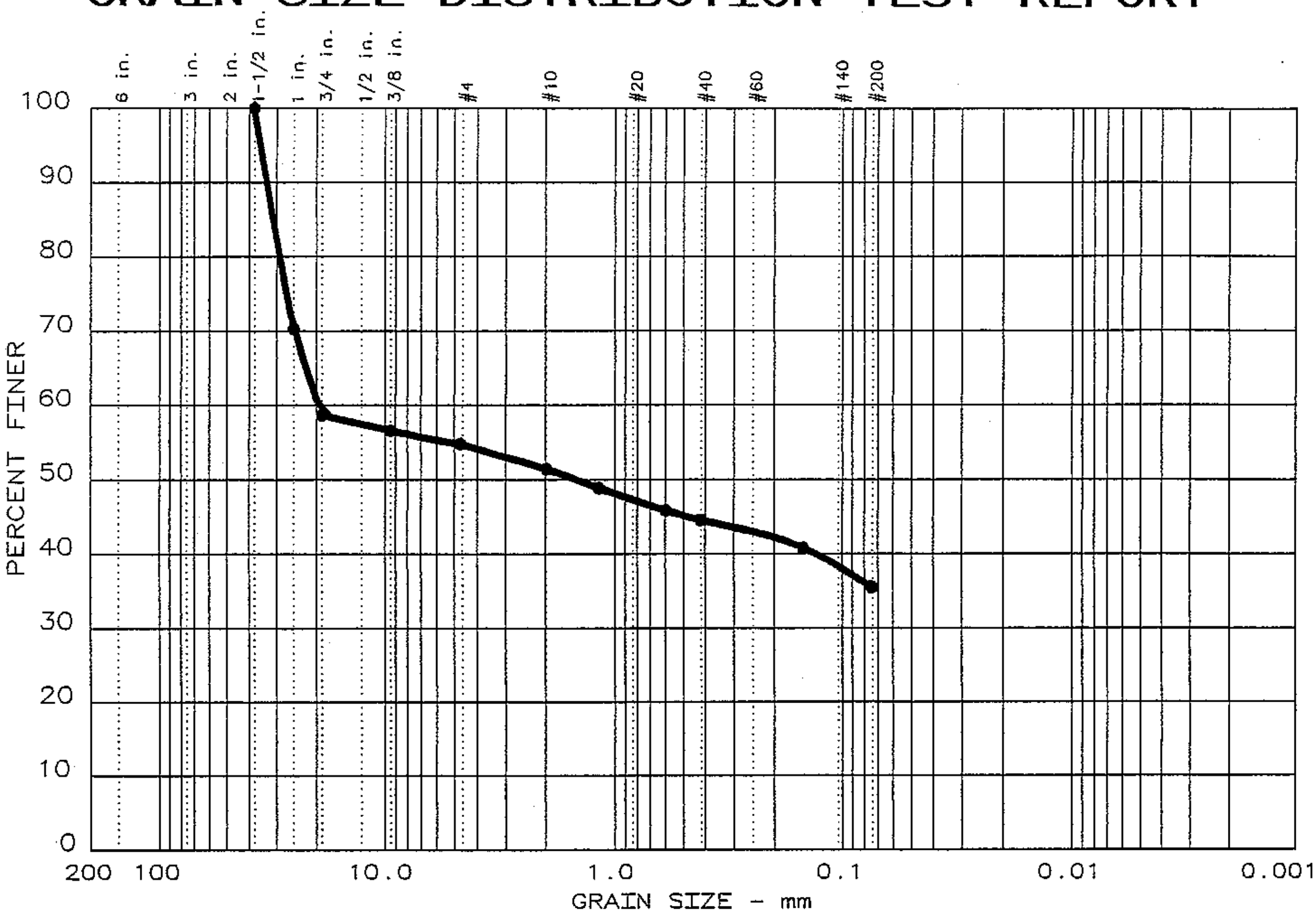
COBBLES	GRAVEL		SAND			SILT OR CLAY	USCS
	COARSE	FINE	CRS	MEDIUM	FINE		

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	WENTWORTH
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			

Client: Goodson & Associates Boring No.: BR-5 Sample No.:
Job Number: 2014-106 Depth: 38.0-39.5
Classification: **Classification Not Performed**

Advanced Terra Testing, Inc.

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	45.2	19.3	35.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	43	27	31.2	19.7	1.48					

MATERIAL DESCRIPTION	USCS	AASHTO
● Clayey Gravel with Sand	GC	A-7-6(4)

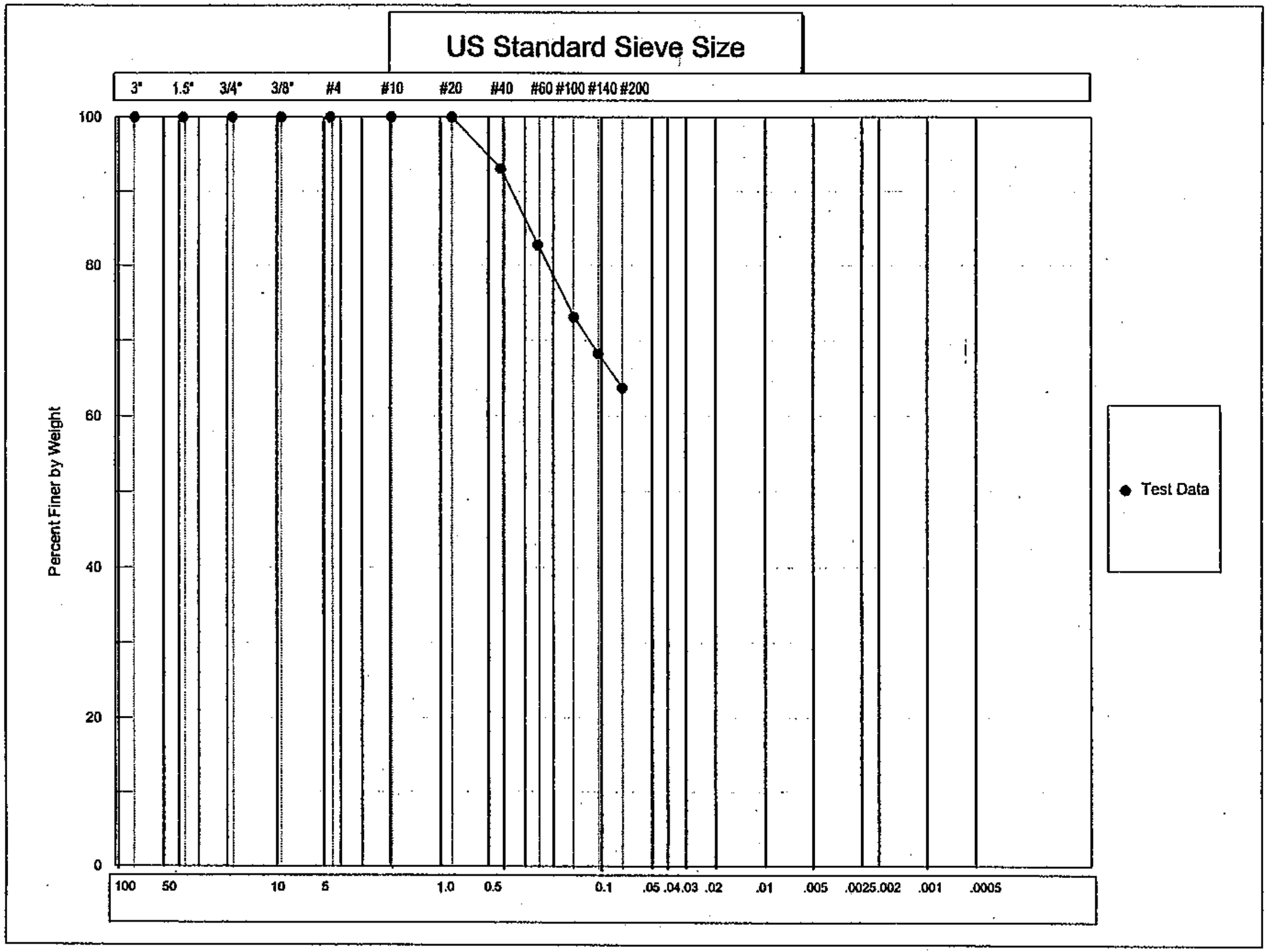
Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-6 @ 5'

Date: 08.03.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______



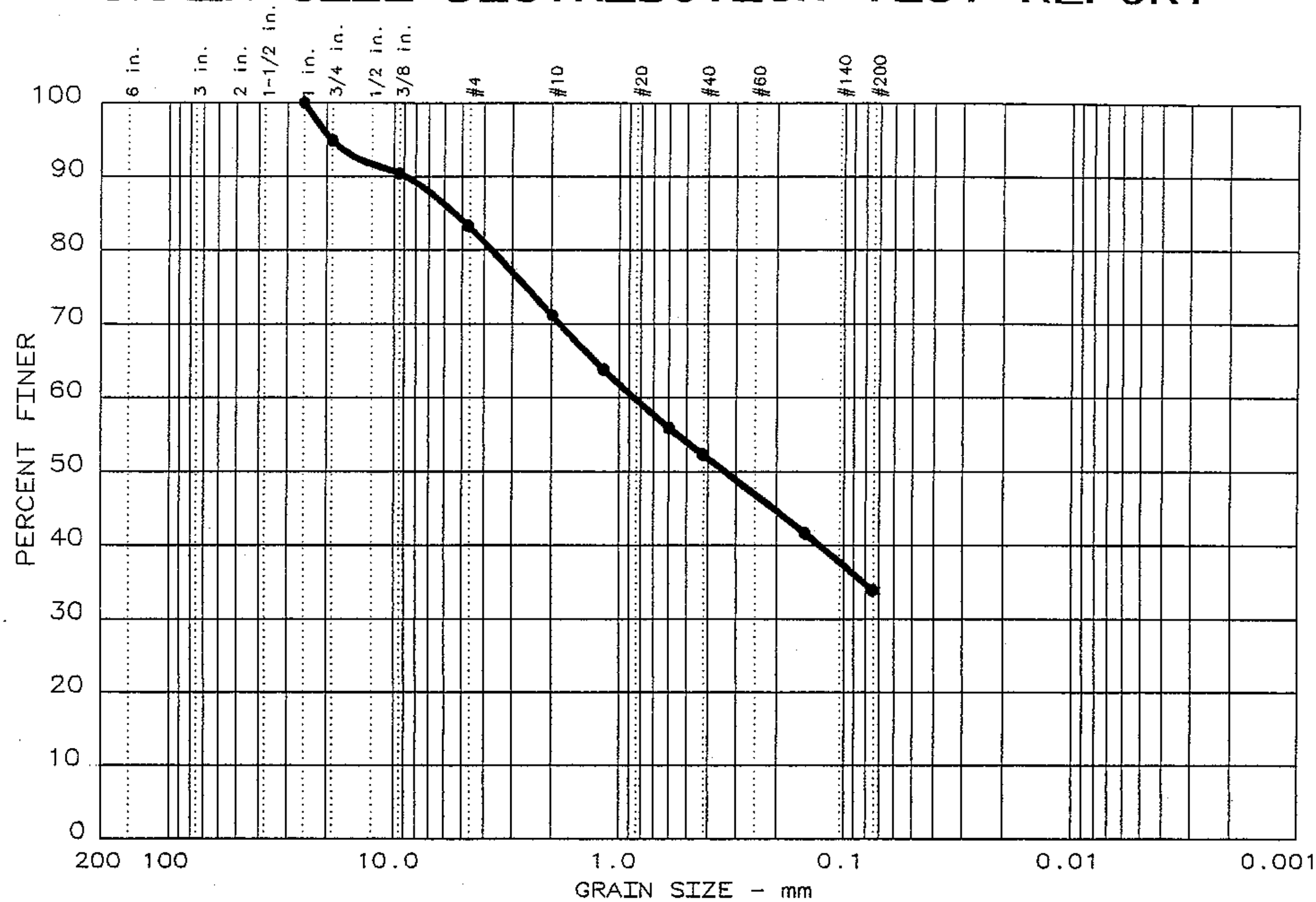
COBBLES	GRAVEL		SAND			SILT OR CLAY	USCS
	COARSE	FINE	CRS	MEDIUM	FINE		

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	WENTWORTH
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			

Client: Goodson & Associates Boring No.: BR-6 Sample No.:
Job Number: 2014-105 Depth: 28.5-30.5'
Classification: **Classification Not Performed**

Advanced Terra Testing, Inc.

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	16.7	49.4	33.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	5.37	0.861	0.335					

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Sand with Gravel	SM	A-2-4(0)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-7 @ 10'

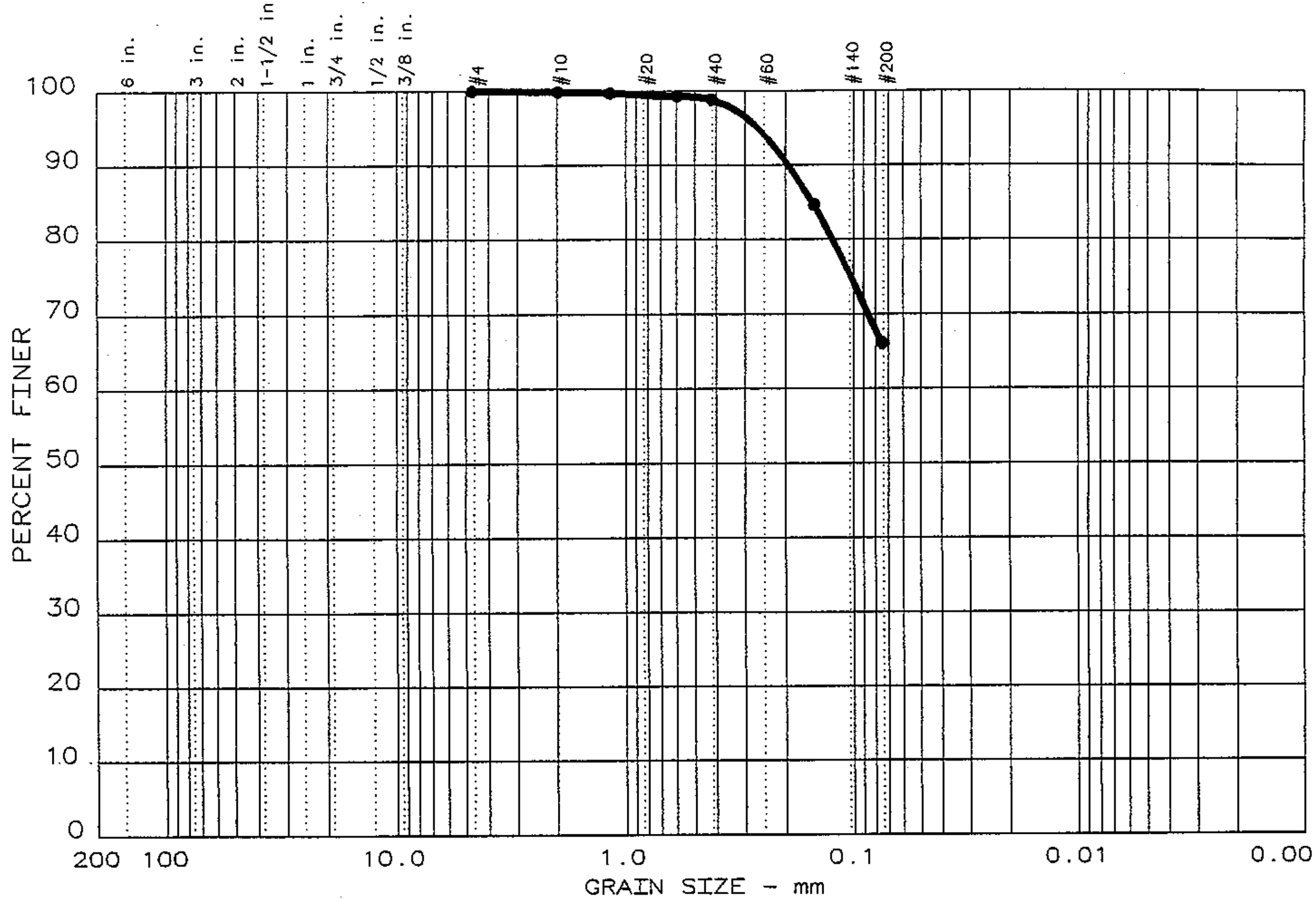
Date: 08.03.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	33.9	66.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	0.151							

MATERIAL DESCRIPTION	USCS	AASHTO
● Sandy Silt	ML	A-4(0)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-7 @ 15'

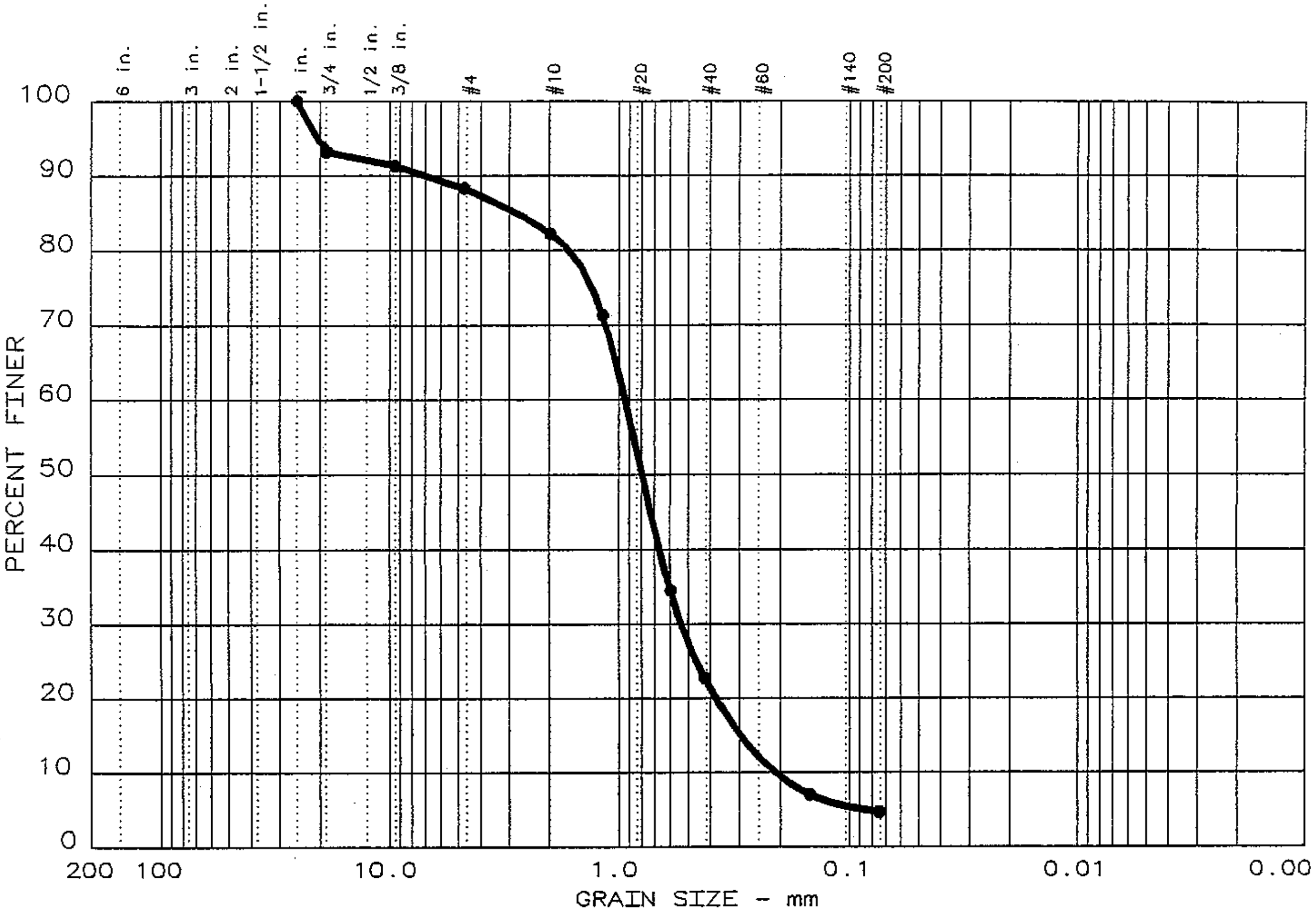
Date: 08.03.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	11.7	83.6	4.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	2.82	0.940	0.794	0.537	0.295	0.209	1.47	4.5

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Sand	SP	A-1-b

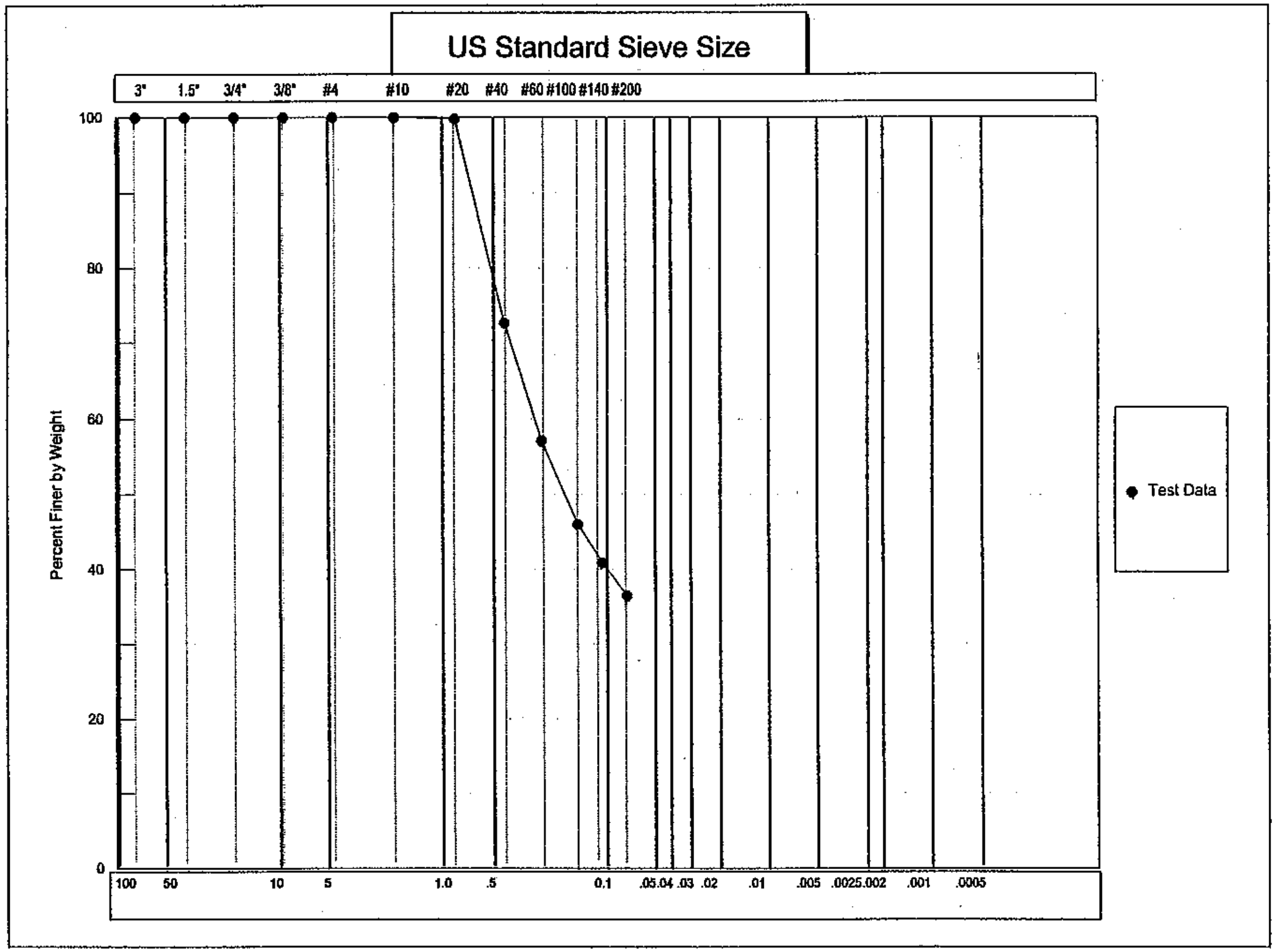
Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-7 @ 30'

Date: 08.03.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______



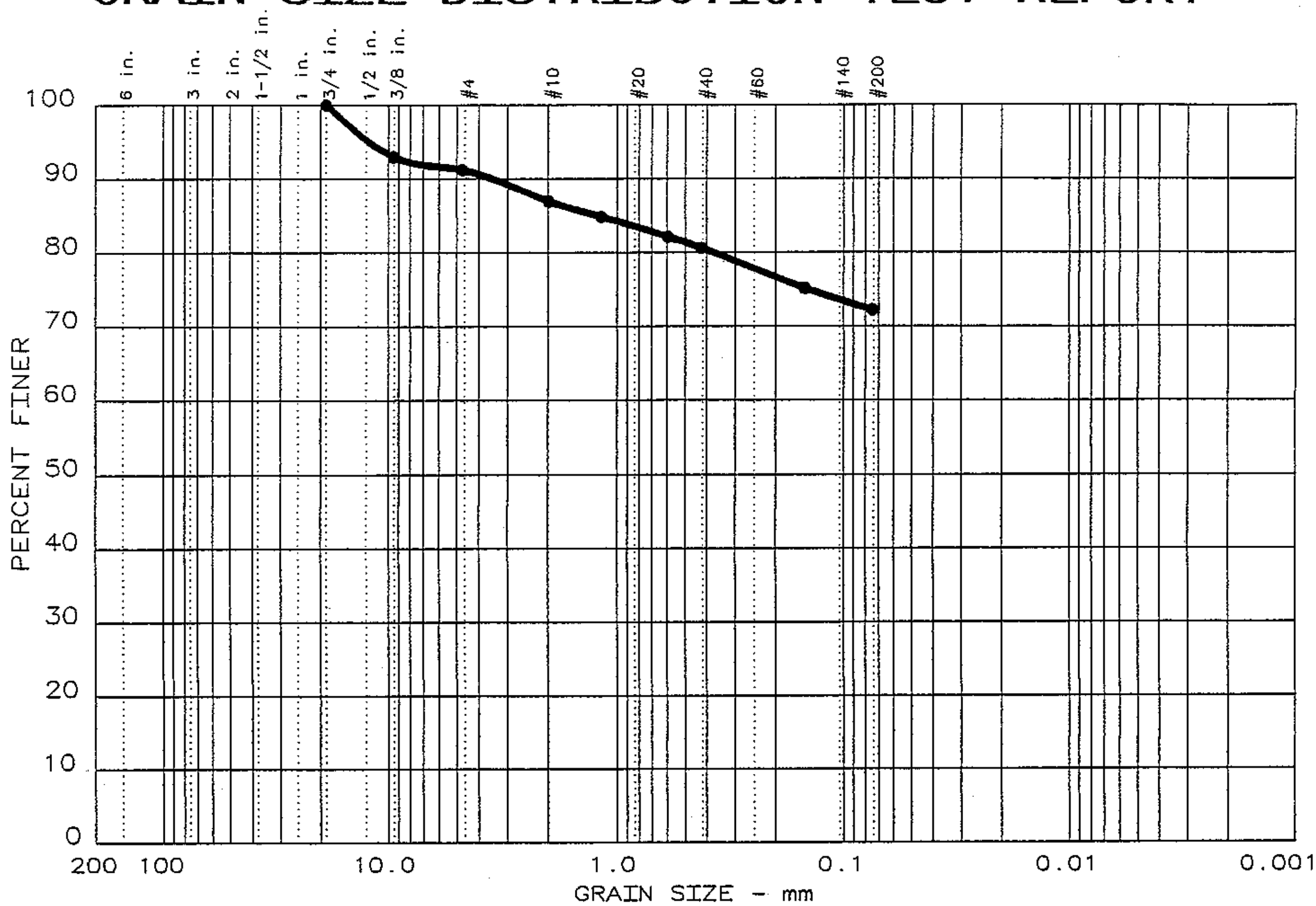
COBBLES	GRAVEL		SAND			SILT OR CLAY	USCS
	COARSE	FINE	CRS	MEDIUM	FINE		

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	WENTWORTH
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			

Client: Goodson & Associates Boring No.: BR-7 Sample No.:
Job Number: 2014-105 Depth: 55-55.6'
Classification____________

Advanced Terra Testing, Inc.

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	8.9	18.9	72.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	40	19	1.23							

MATERIAL DESCRIPTION	USCS	AASHTO
● Lean Clay with Sand	CL	A-6(13)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-8 @ 1.5'

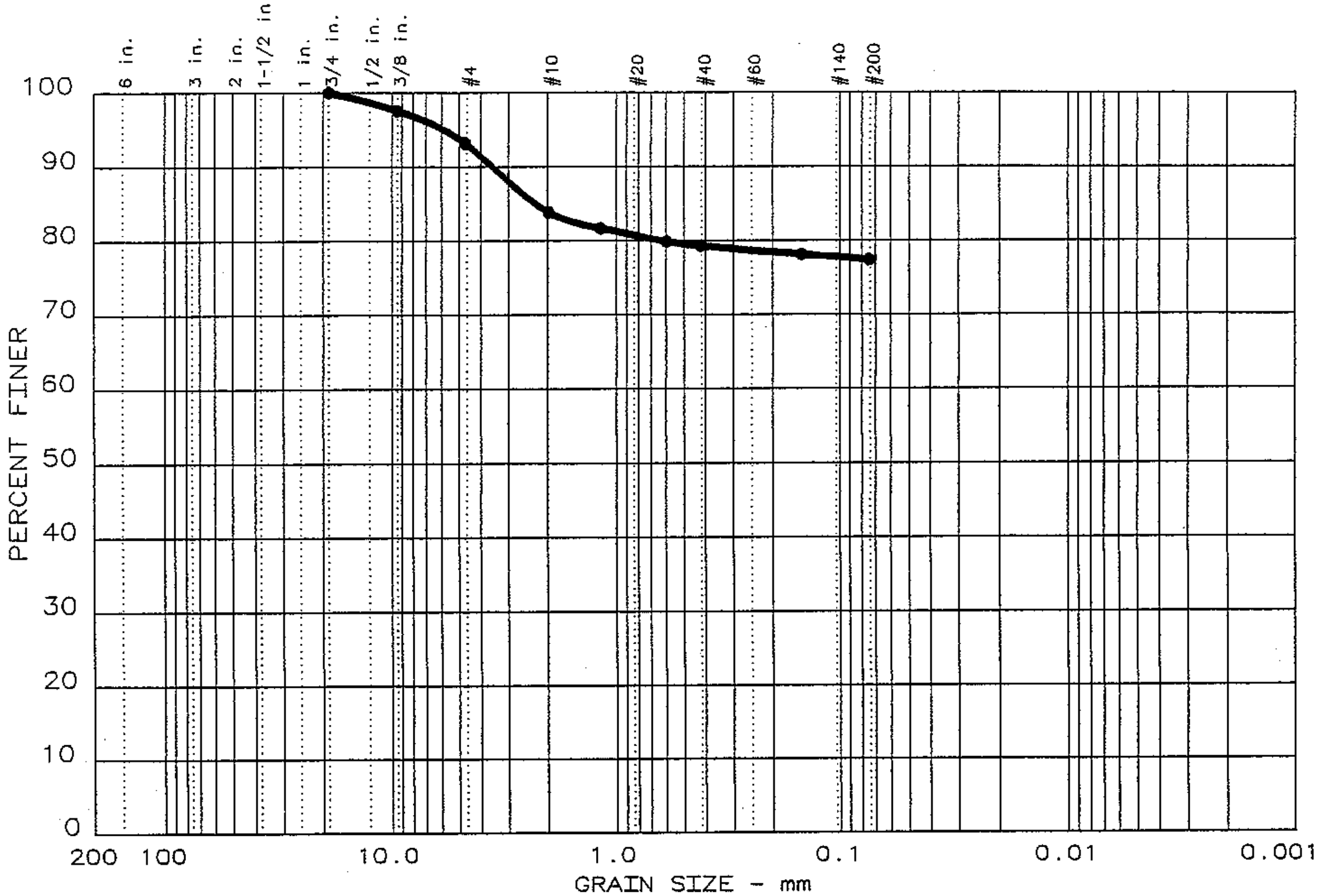
Date: 08.03.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	6.8	15.8	77.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	45	20	2.29							

MATERIAL DESCRIPTION	USCS	AASHTO
● Lean Clay with Sand	CL	A-7-6(16)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-8 @ 10'

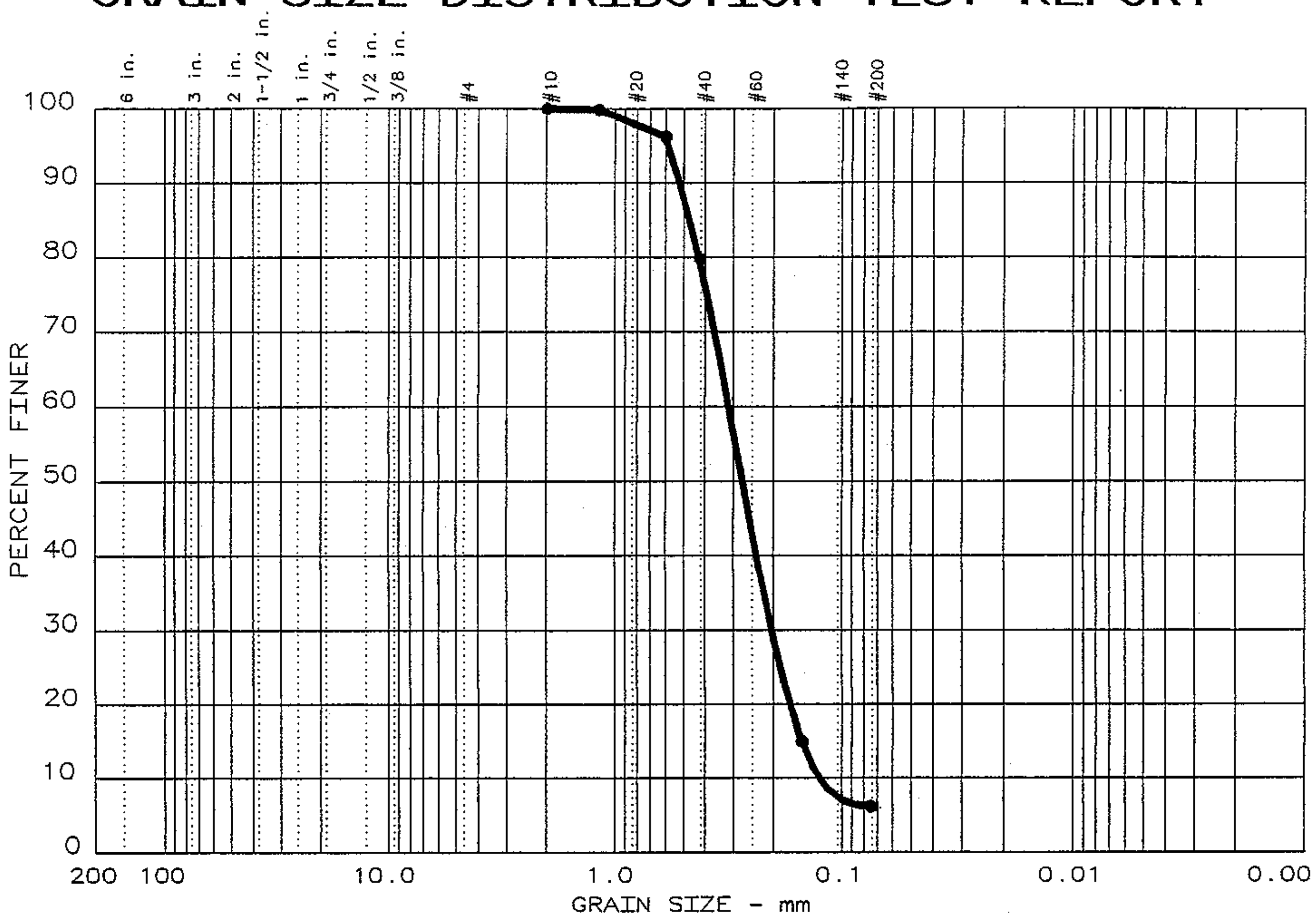
Date: 08.03.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	93.8	6.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	0.470	0.316	0.274	0.203	0.150	0.125	1.05	2.5

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Sand with Silt	SP-SM	A-3

Project No.: 65569.01
Project: 4th Street Bridge
● Location: BR-8 @ 30'

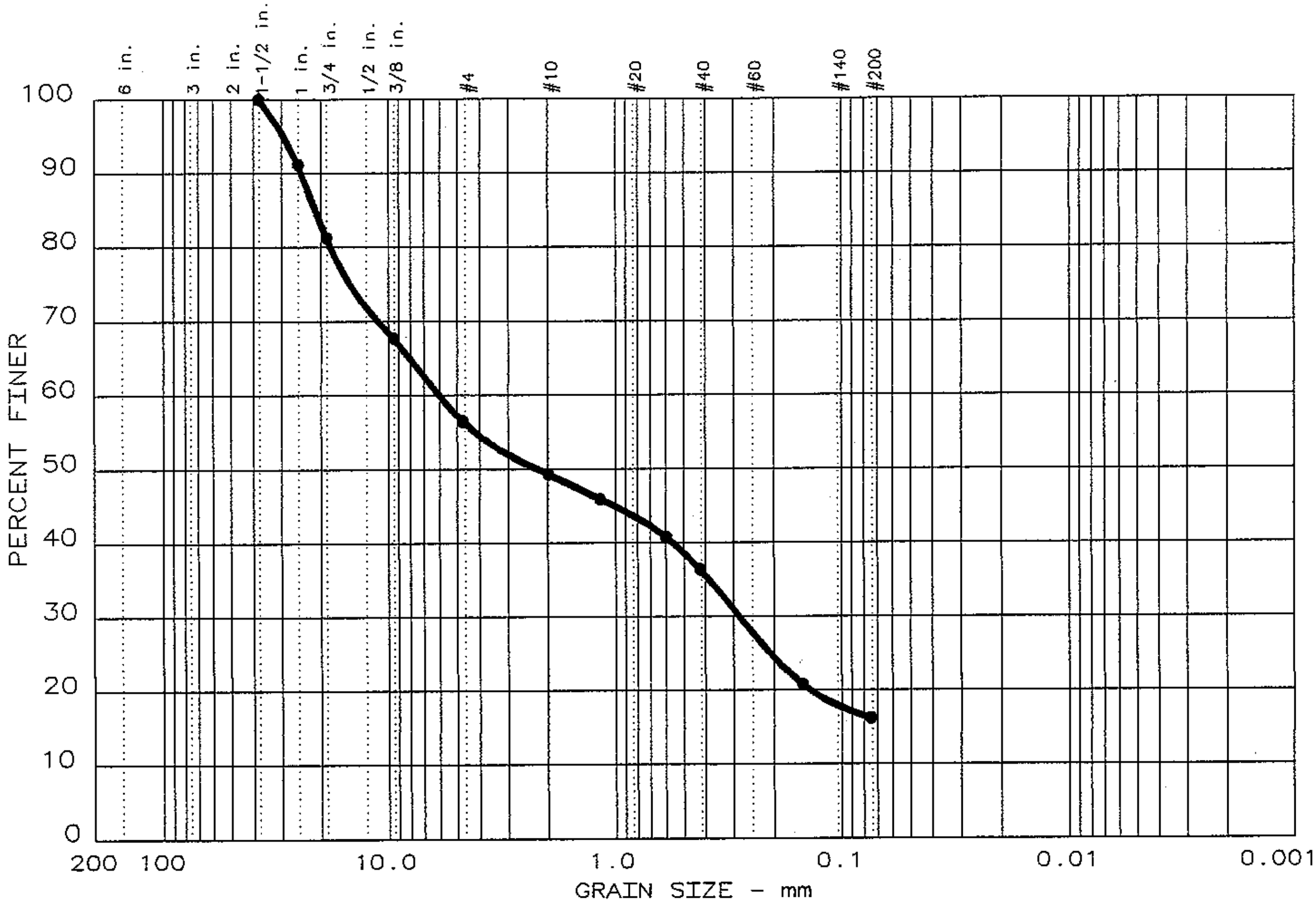
Date: 08.03.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	43.5	40.3	16.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	21.3	5.99	2.25	0.283				

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Gravel with Sand	GM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DC-1 @ 0'-5'

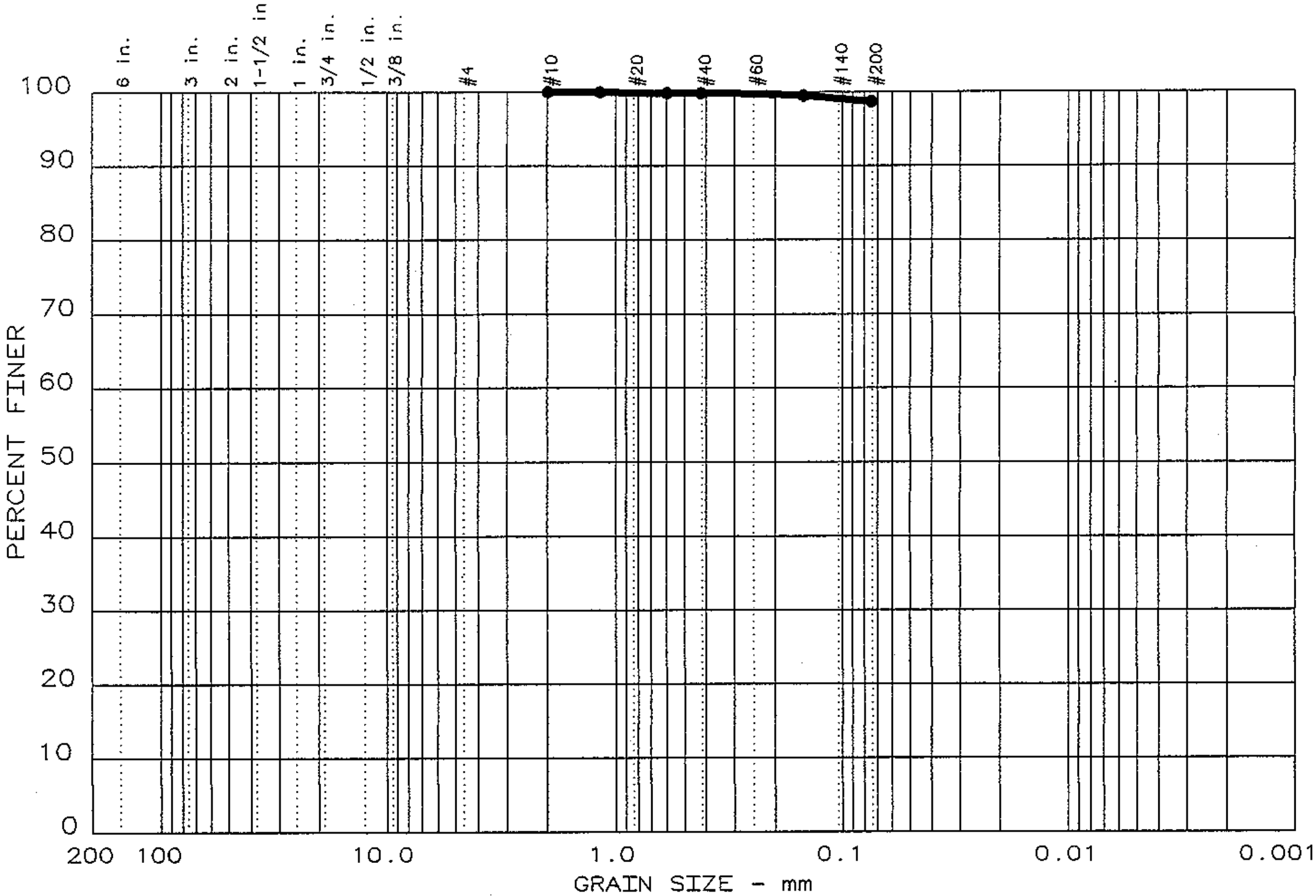
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	1.3	98.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	48	23								

MATERIAL DESCRIPTION	USCS	AASHTO
● Lean Clay	CL	A-7-6(26)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DC-1 @ 15'

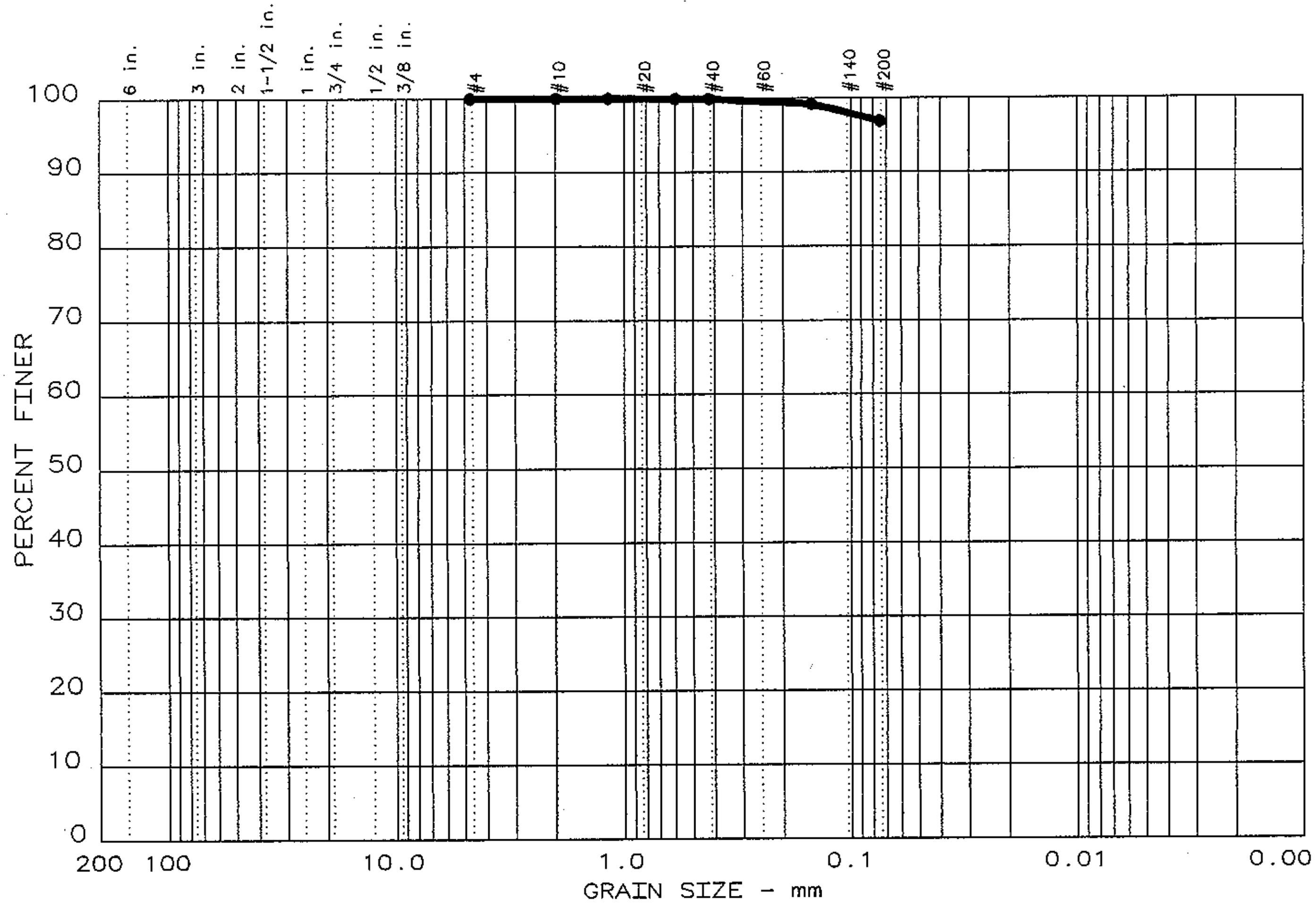
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	3.2	96.8	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	41	20								

MATERIAL DESCRIPTION	USCS	AASHTO
● Lean Clay	CL	A-7-6(21)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-1 @ 1.5'

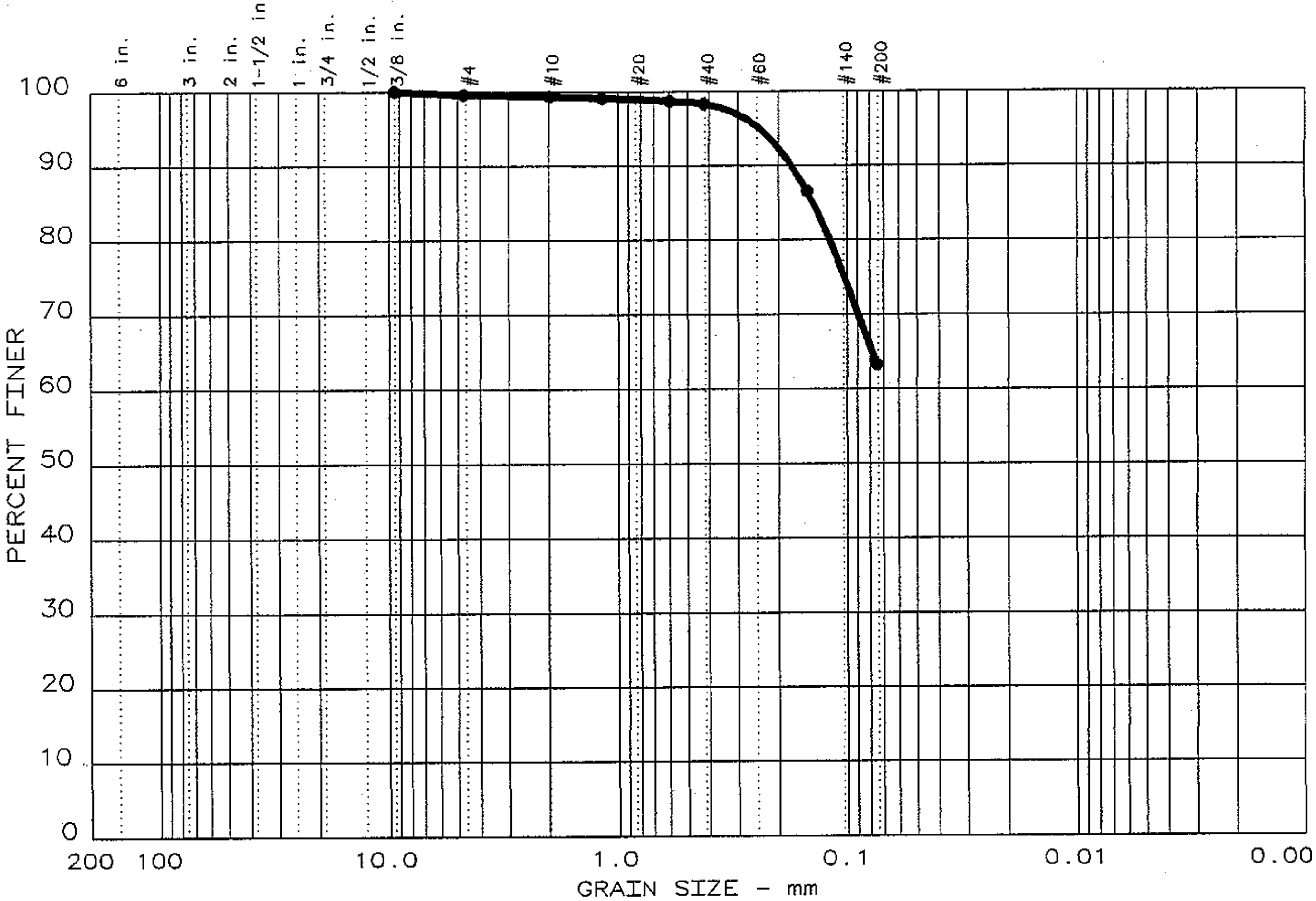
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.4	36.2	63.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	28	12	0.141							

MATERIAL DESCRIPTION	USCS	AASHTO
● Sandy Lean Clay	CL	A-6(5)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-2 @ 5'

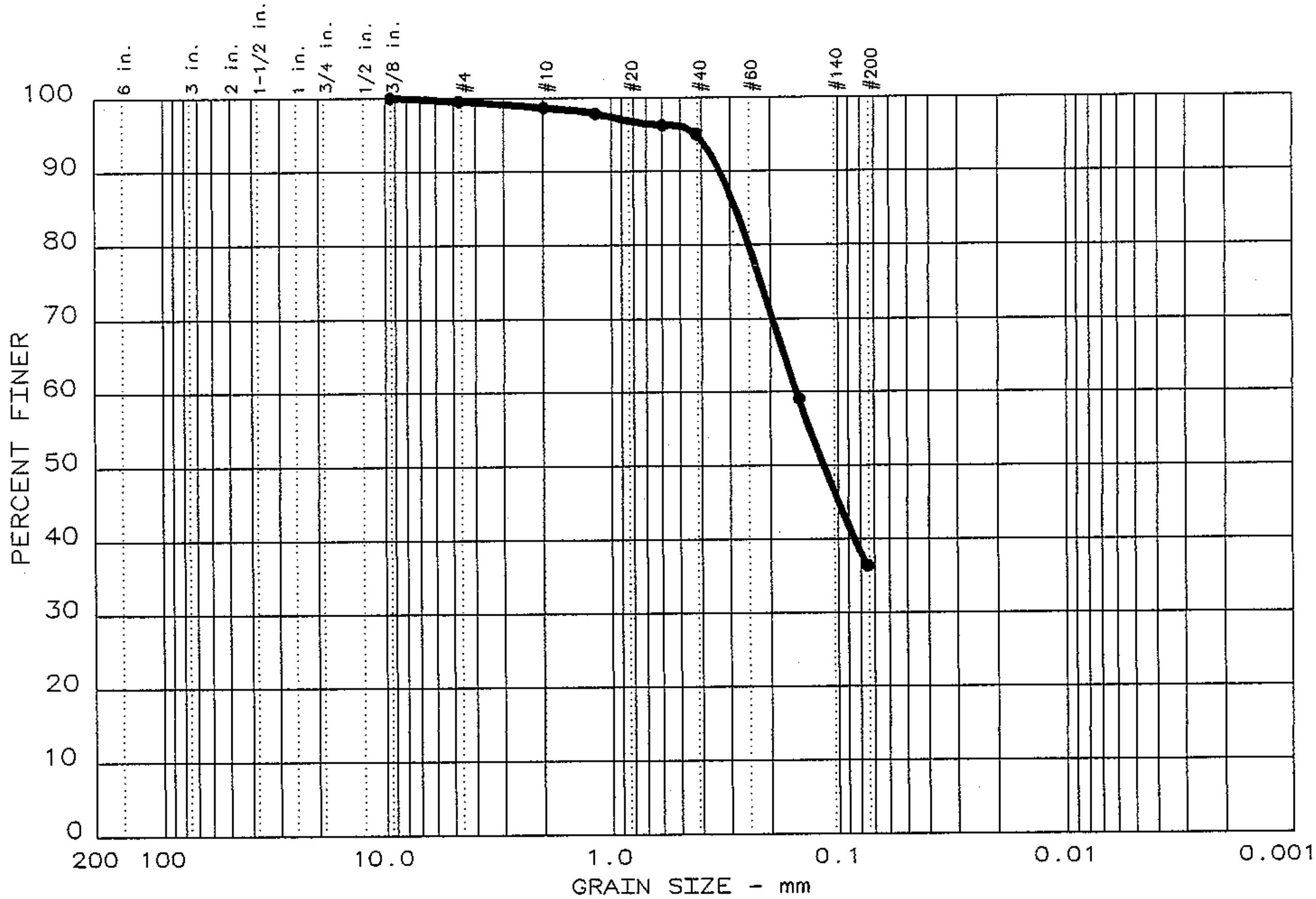
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.5	63.2	36.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	0.284	0.154	0.117					

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Sand	SM	A-4(0)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-2 @ 15'

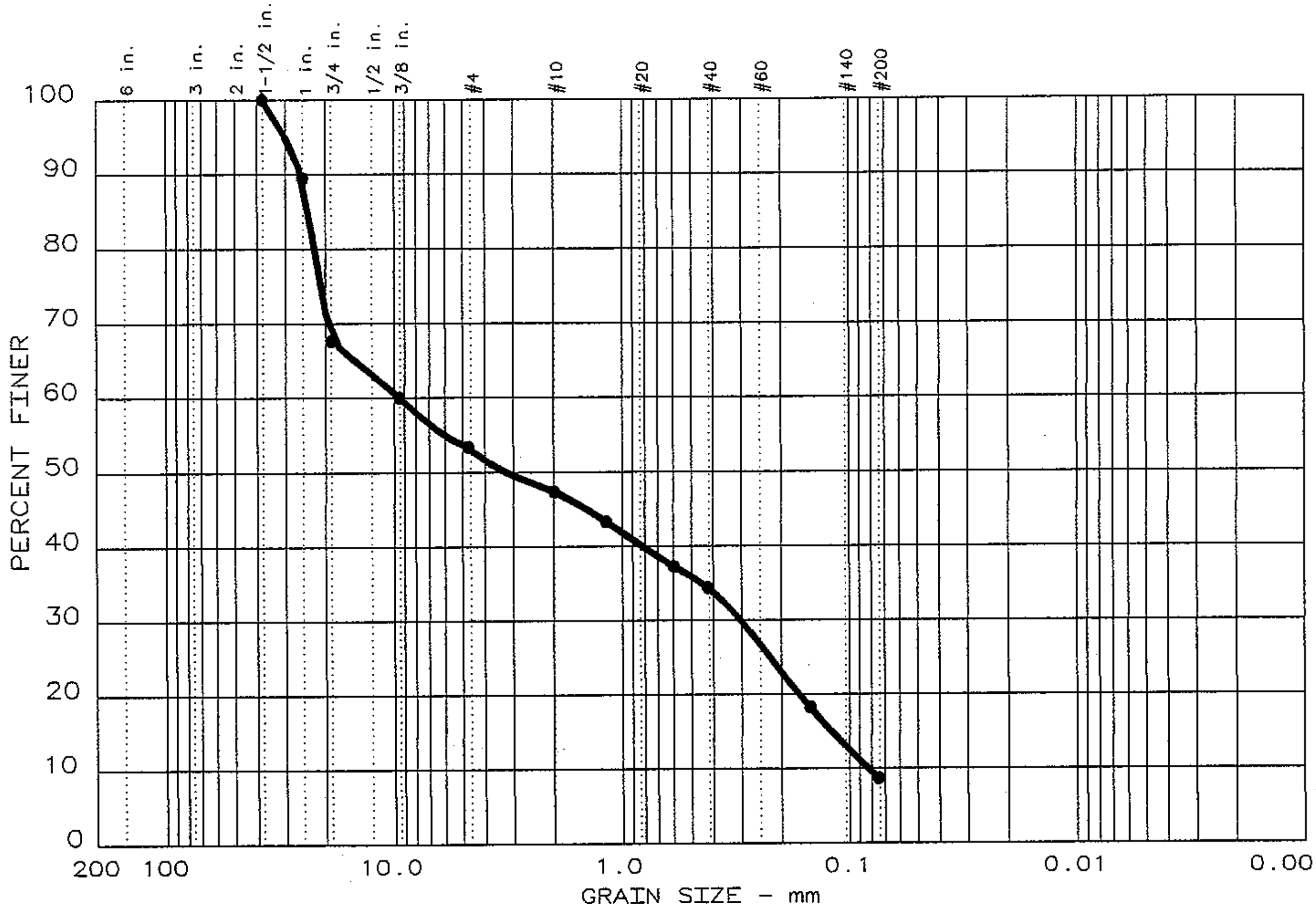
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	46.6	44.7	8.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	23.9	9.42	3.23	0.301	0.120	0.0830	0.12	113.5

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Gravel with Silt and Sand	GP-GM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-2 @ 20'

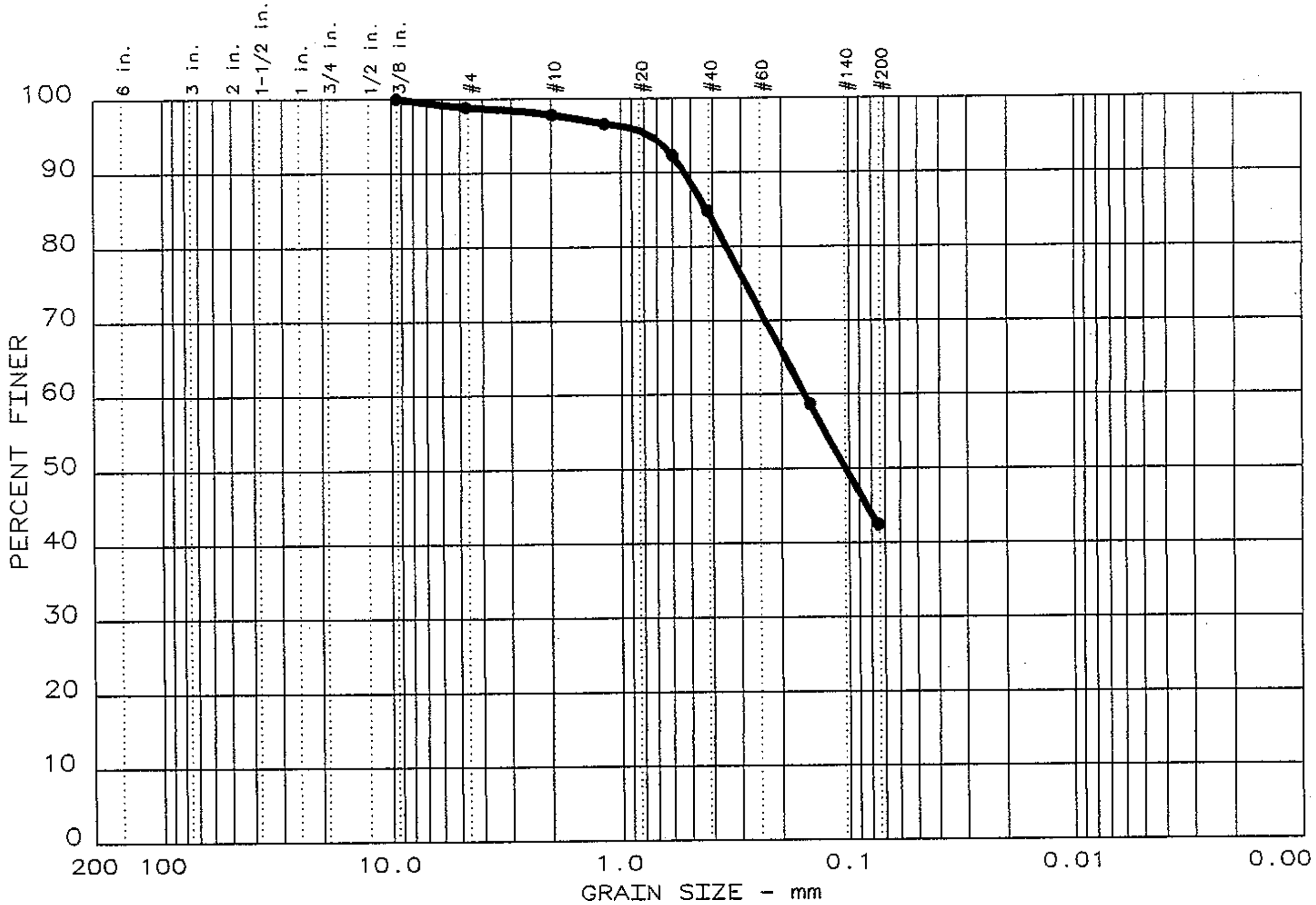
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	1.2	56.4	42.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	24	6	0.427	0.158	0.103					

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Clayey Sand	SC-SM	A-4(0)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-3 @ 1.5'

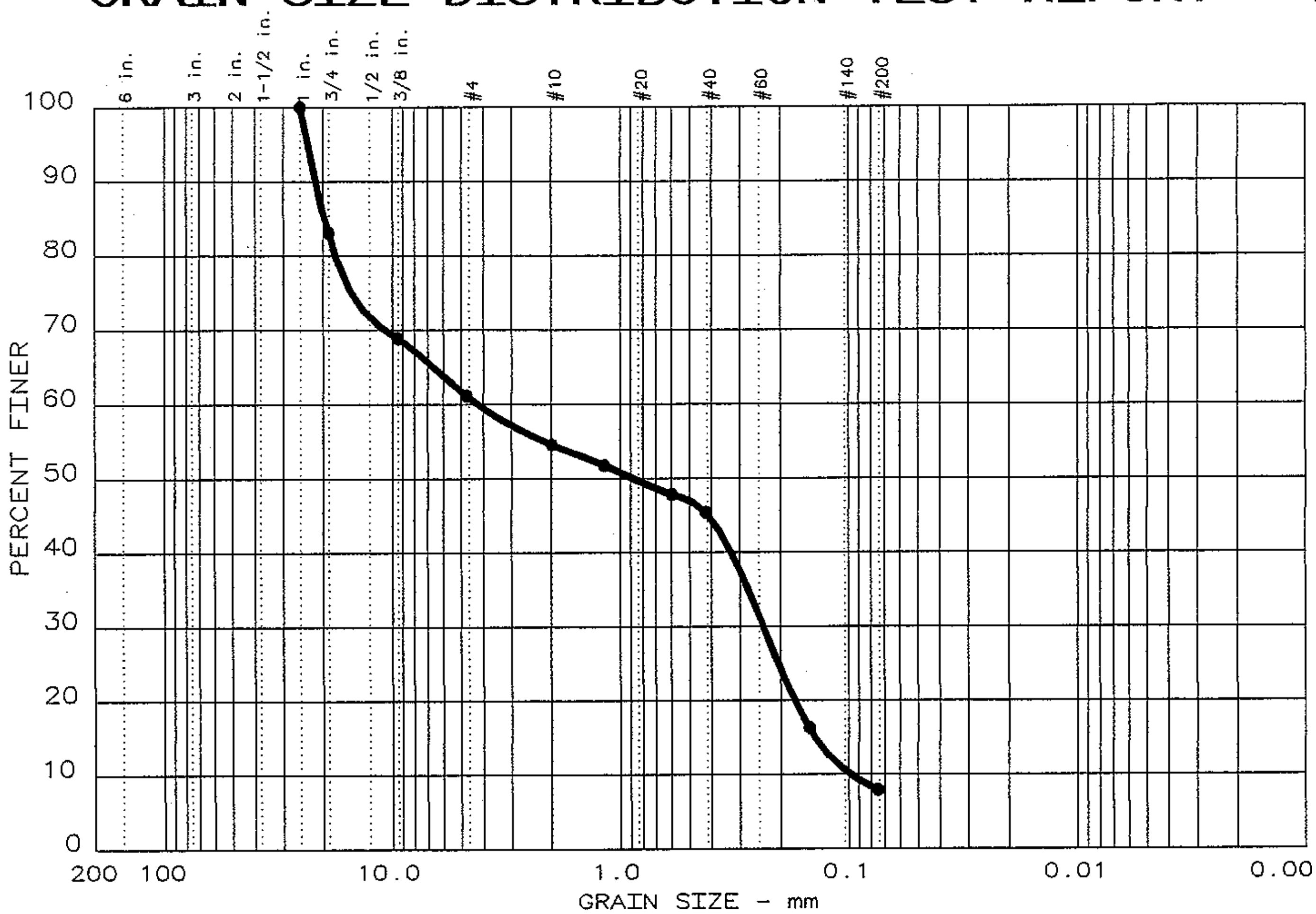
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	38.8	53.3	7.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	19.8	4.18	0.884	0.237	0.140	0.0978	0.14	42.8

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Sand with Silt and Gravel	SP-SM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-3 @ 5'

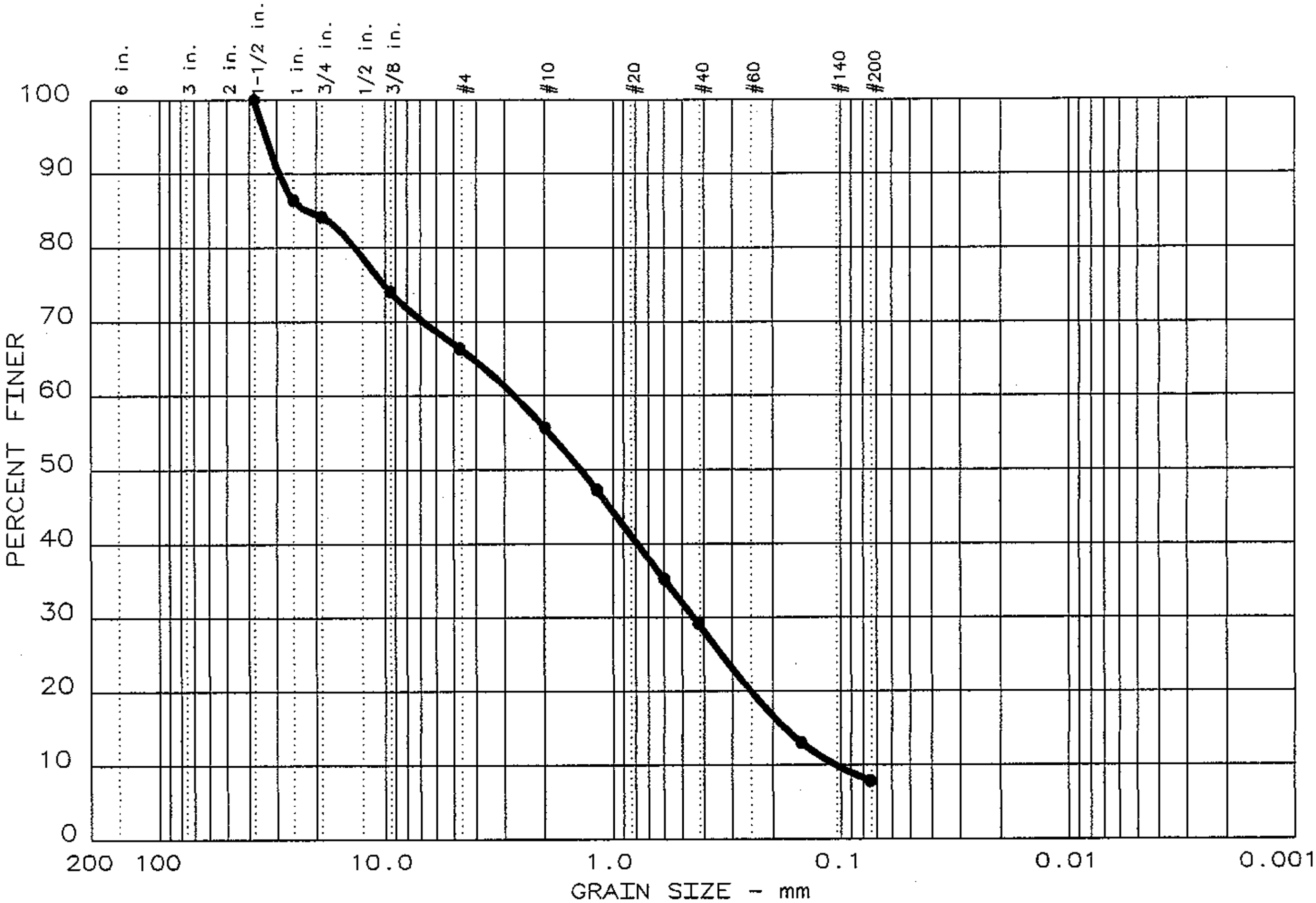
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	33.6	58.6	7.8	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	22.1	2.72	1.38	0.442	0.176	0.106	0.68	25.7

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Sand with Silt and Gravel	SP-SM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-3 @ 15'

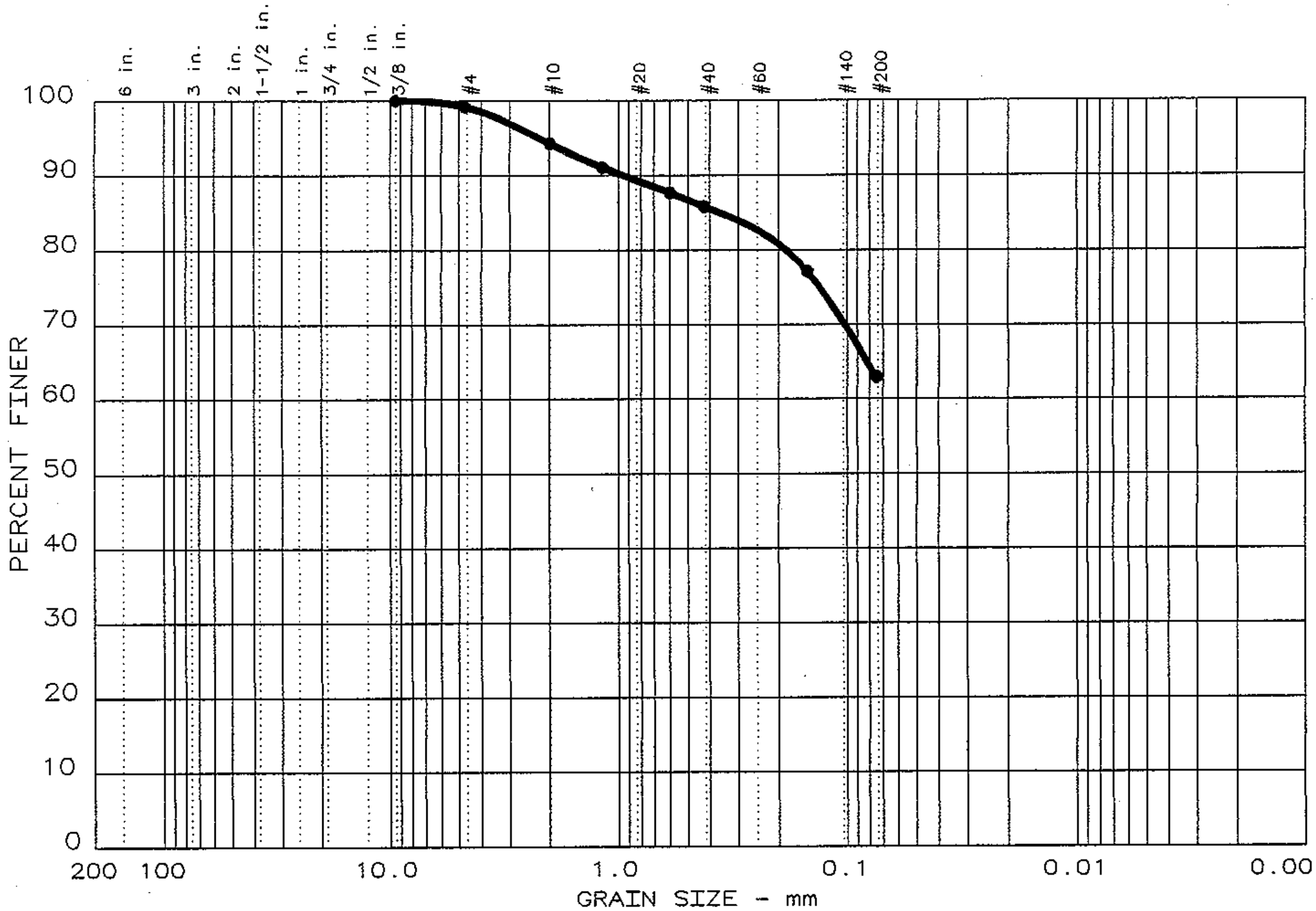
Date: 08.24.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.8	36.2	63.0	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	34	15	0.363							

MATERIAL DESCRIPTION	USCS	AASHTO
● Sandy Lean Clay	CL	A-6(7)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-4 @ 5'

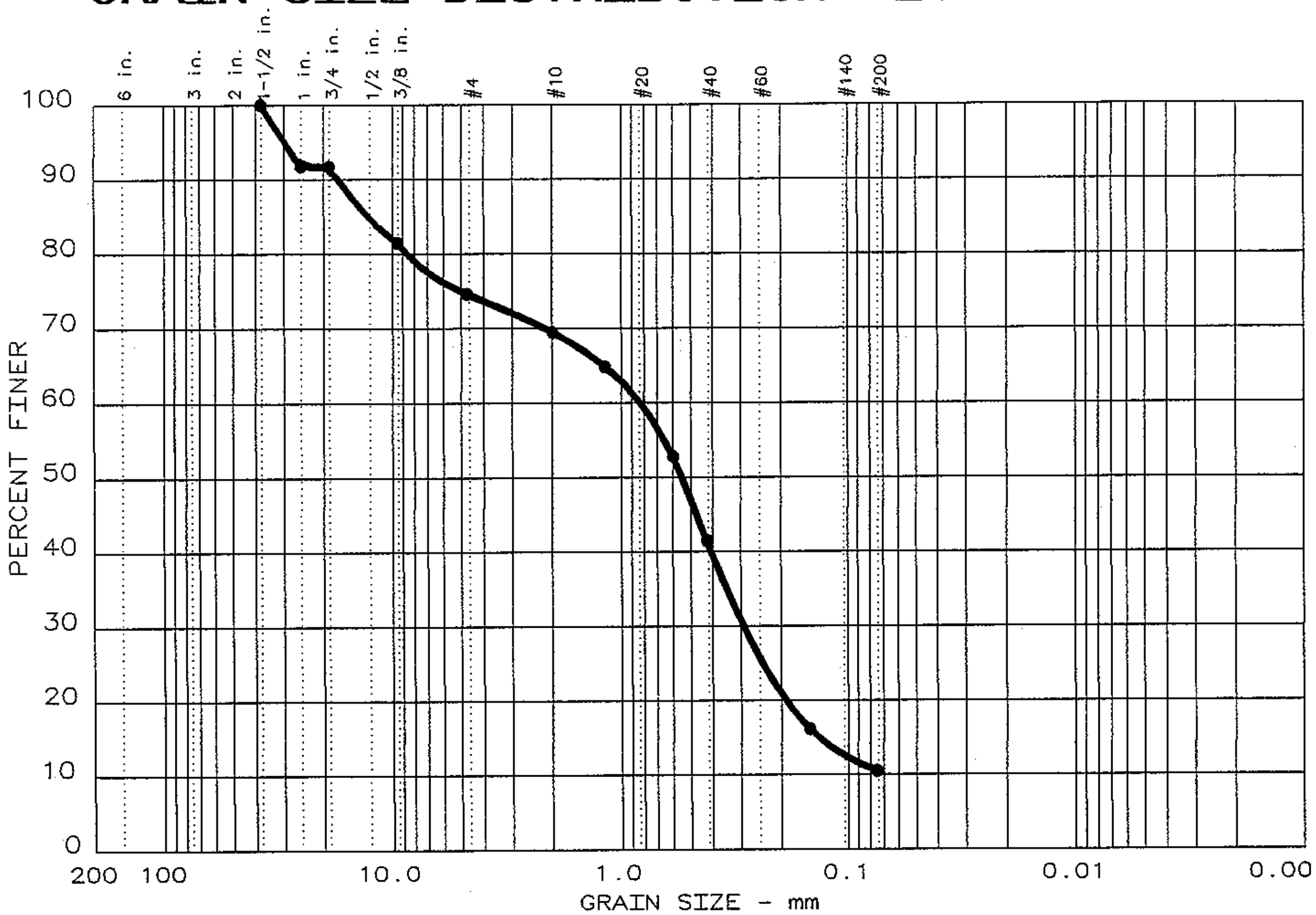
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	25.4	64.1	10.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	12.7	0.822	0.548	0.291	0.136			

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Sand with Silt and Gravel	SP-SM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-4 @ 15'

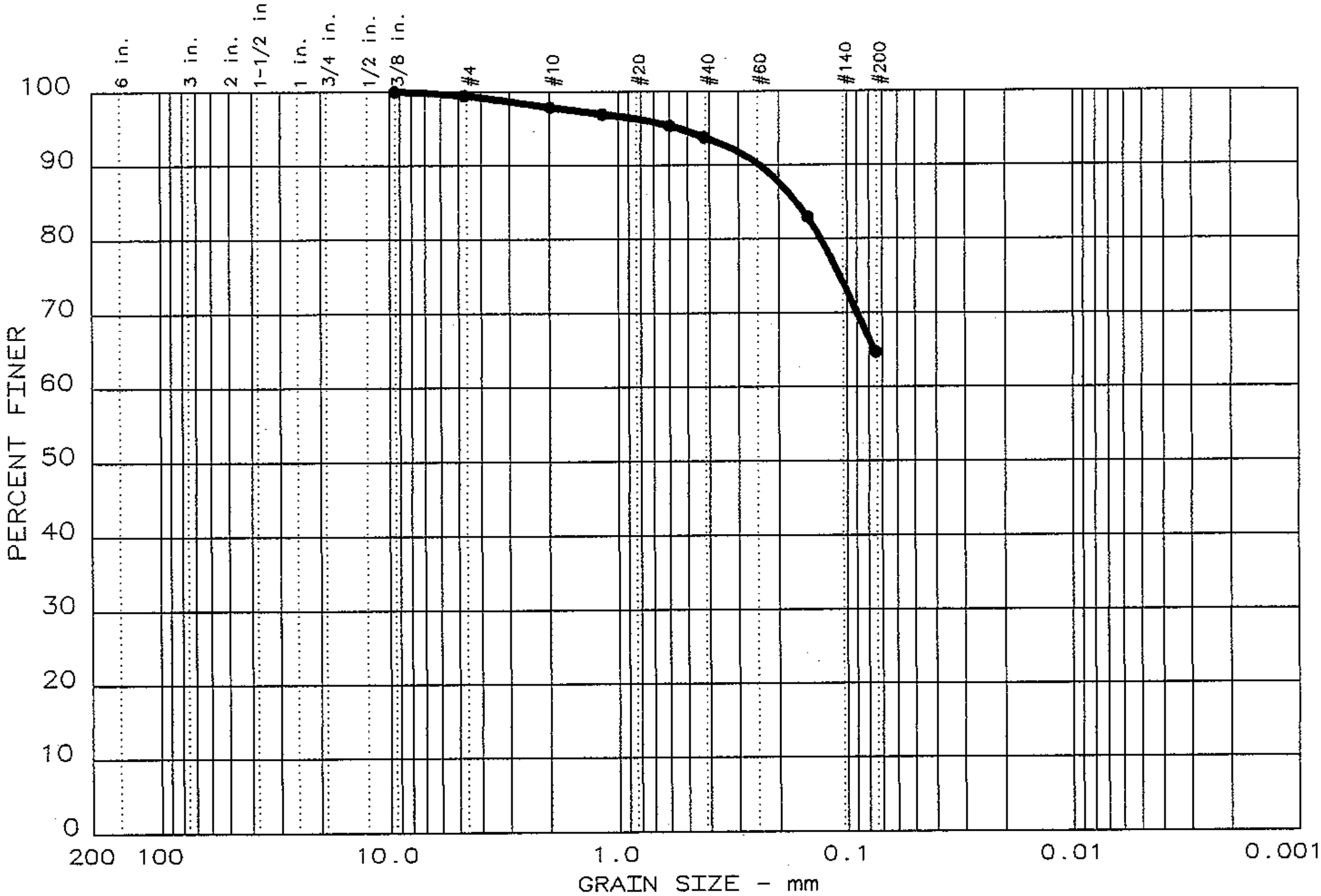
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.5	34.7	64.8	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	30	13	0.166							

MATERIAL DESCRIPTION	USCS	AASHTO
● Sandy Lean Clay	CL	A-6(6)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-5 @ 1.5'

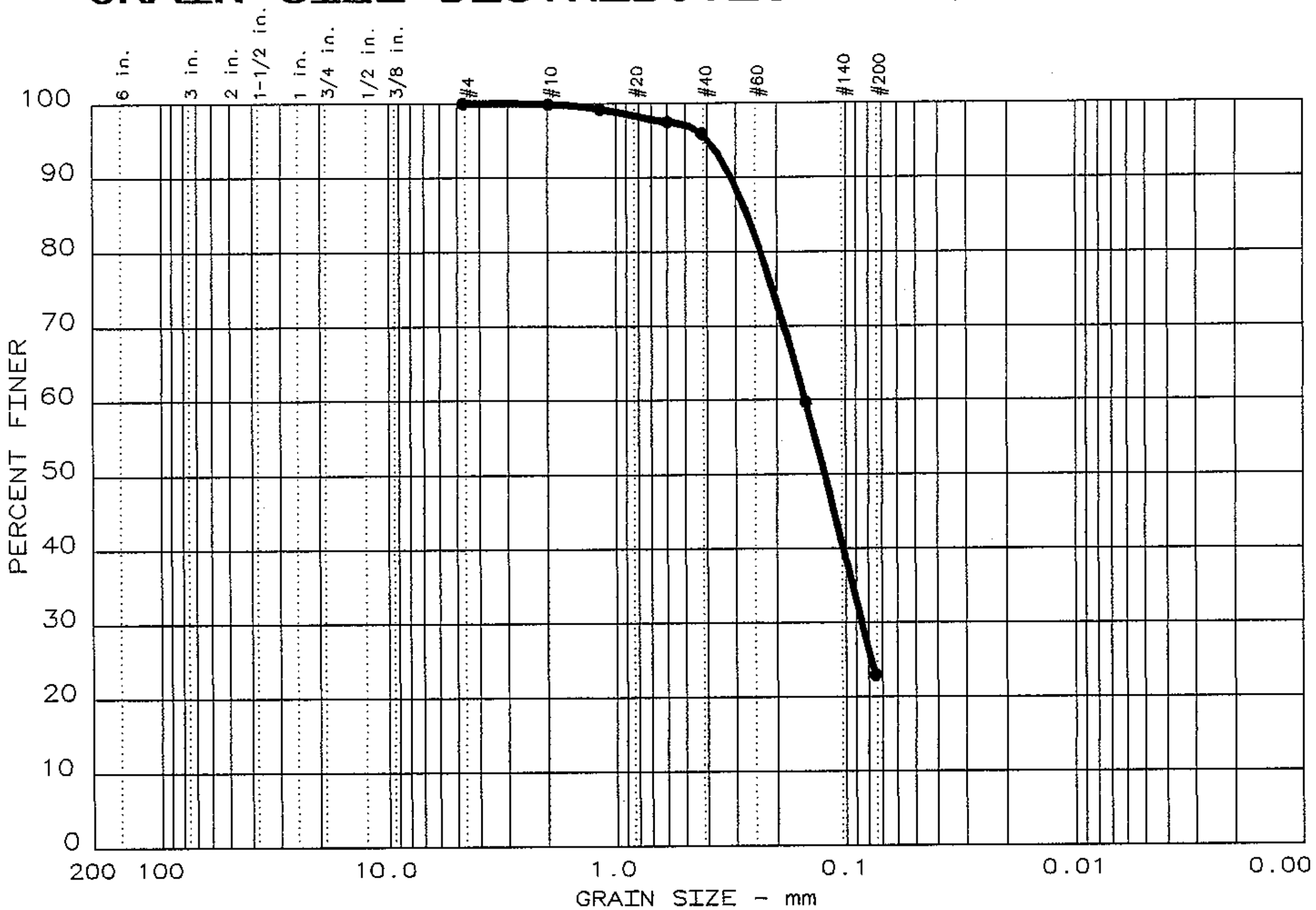
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	77.1	22.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	0.268	0.151	0.124	0.0853				

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Sand	SM	A-2-4(0)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-5 @ 5'

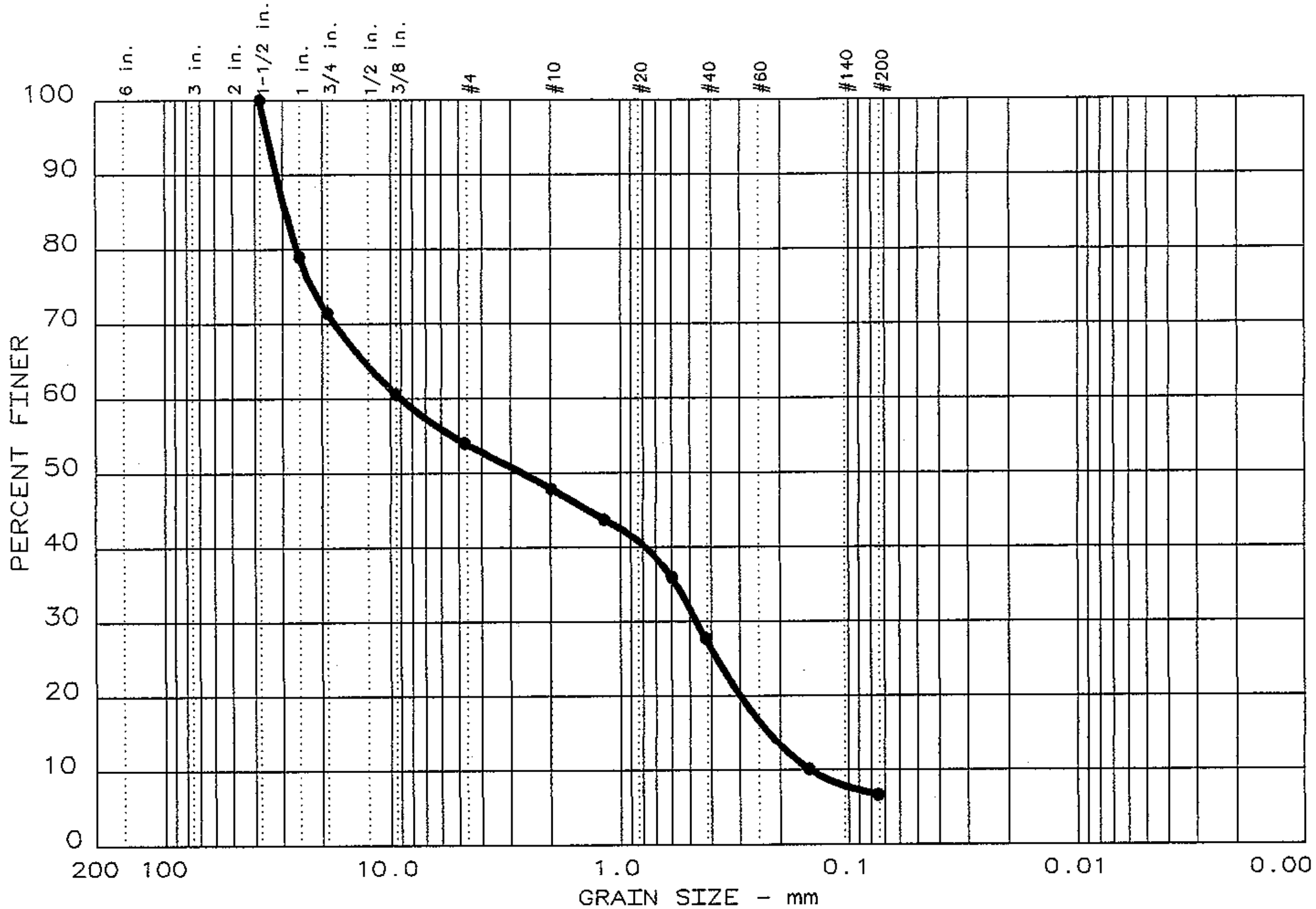
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	46.0	47.4	6.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	29.1	8.99	2.65	0.466	0.223	0.147	0.16	61.1

MATERIAL DESCRIPTION	USCS	AASHTO
● Poorly Graded Sand with Silt and Gravel	SP-SM	A-1-a

Project No.: 65569.01
Project: 4th Street Bridge
● Location: DF-5 @ 20'

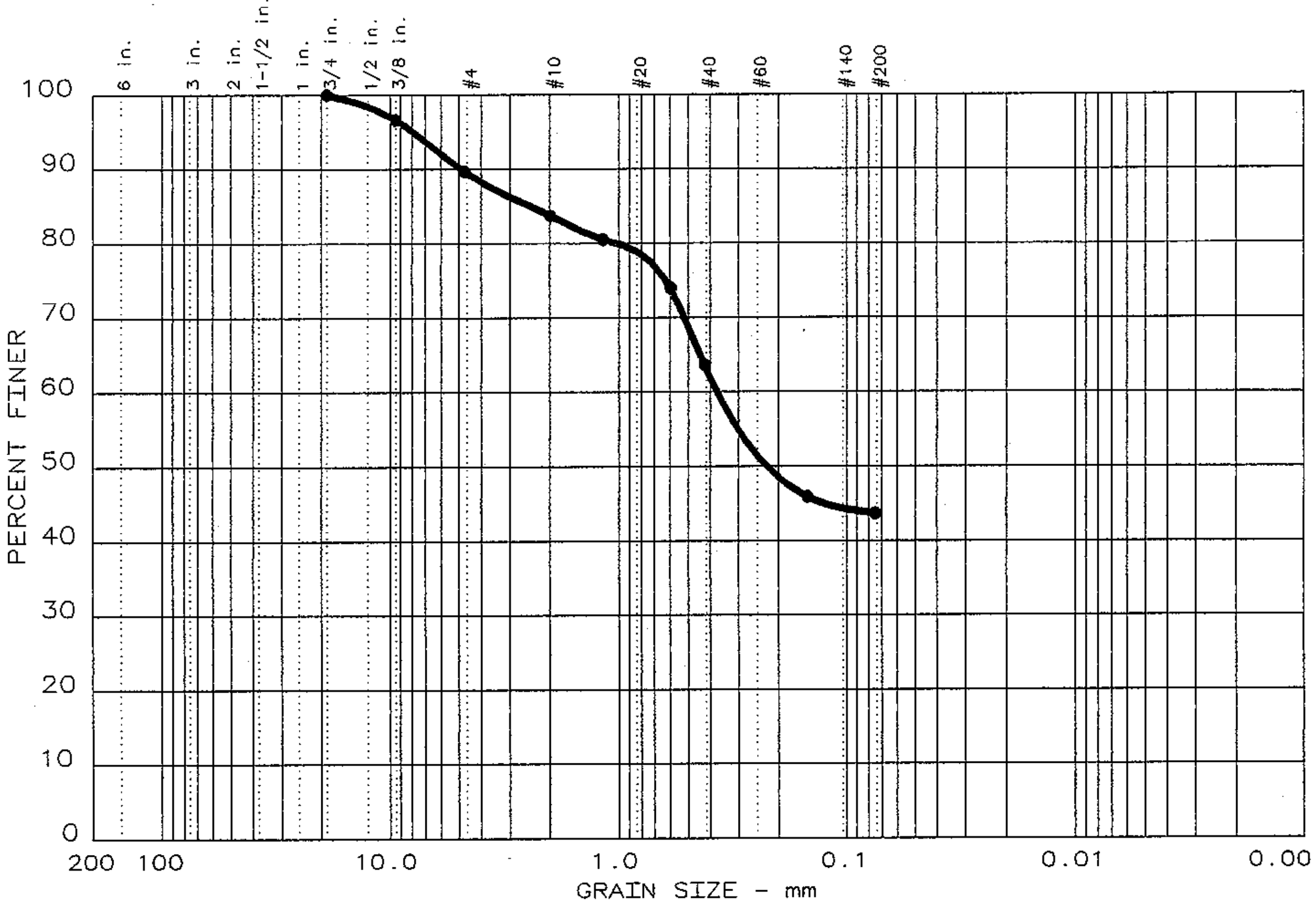
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	10.4	46.0	43.6	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	24	5	2.45	0.372	0.224					

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Clayey Sand	SC-SM	A-4(0)

Project No.: 65569.01
Project: 4th Street Bridge - Pueblo
● Location: P1 @ 1'-5'

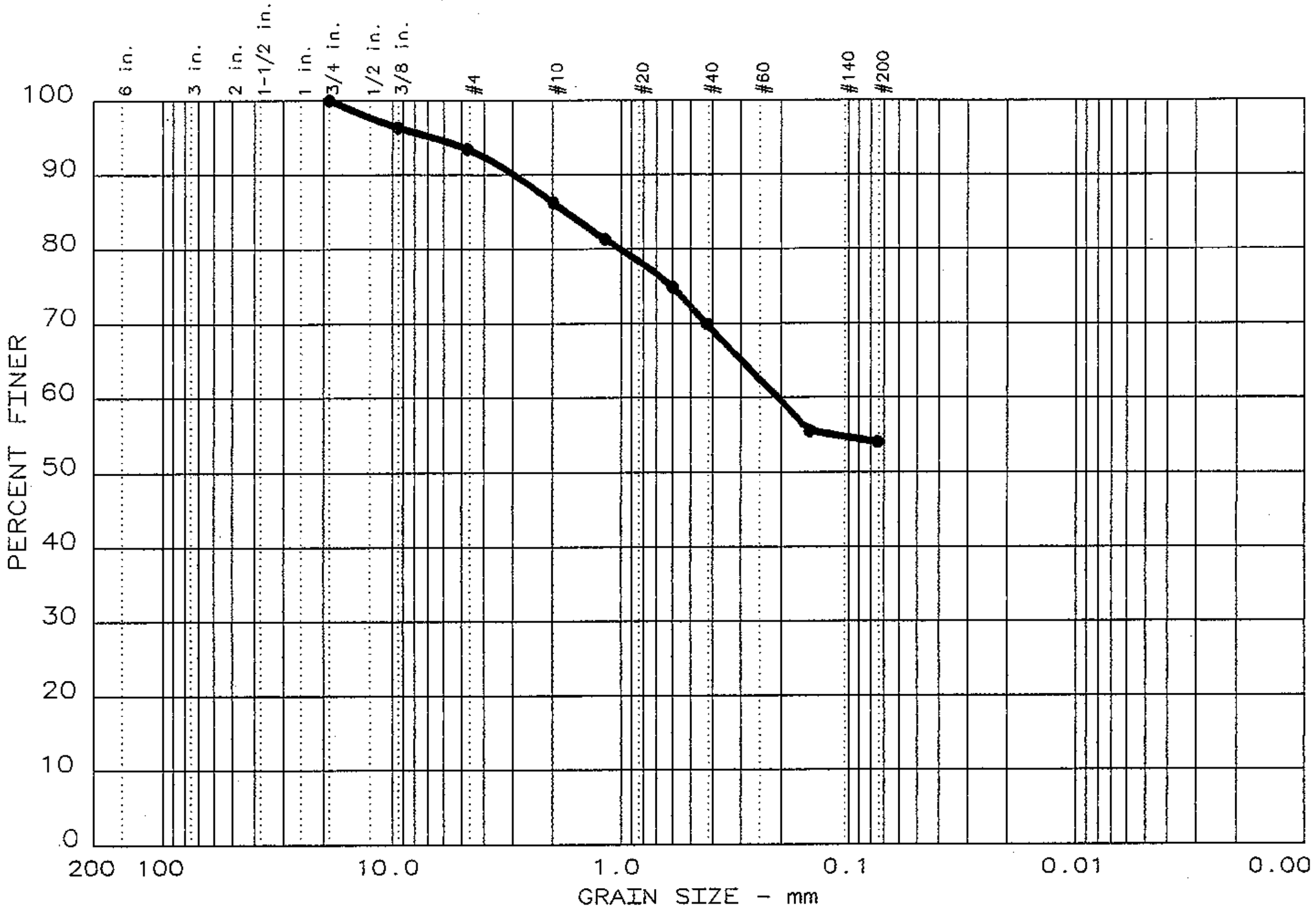
Date: 05.16.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	6.5	39.4	54.1	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	37	16	1.74	0.207						

MATERIAL DESCRIPTION	USCS	AASHTO
● Sandy Lean Clay	CL	A-6(6)

Project No.: 65569.01
Project: 4th Street Bridge - Pueblo
● Location: P2 @ 0'-5'

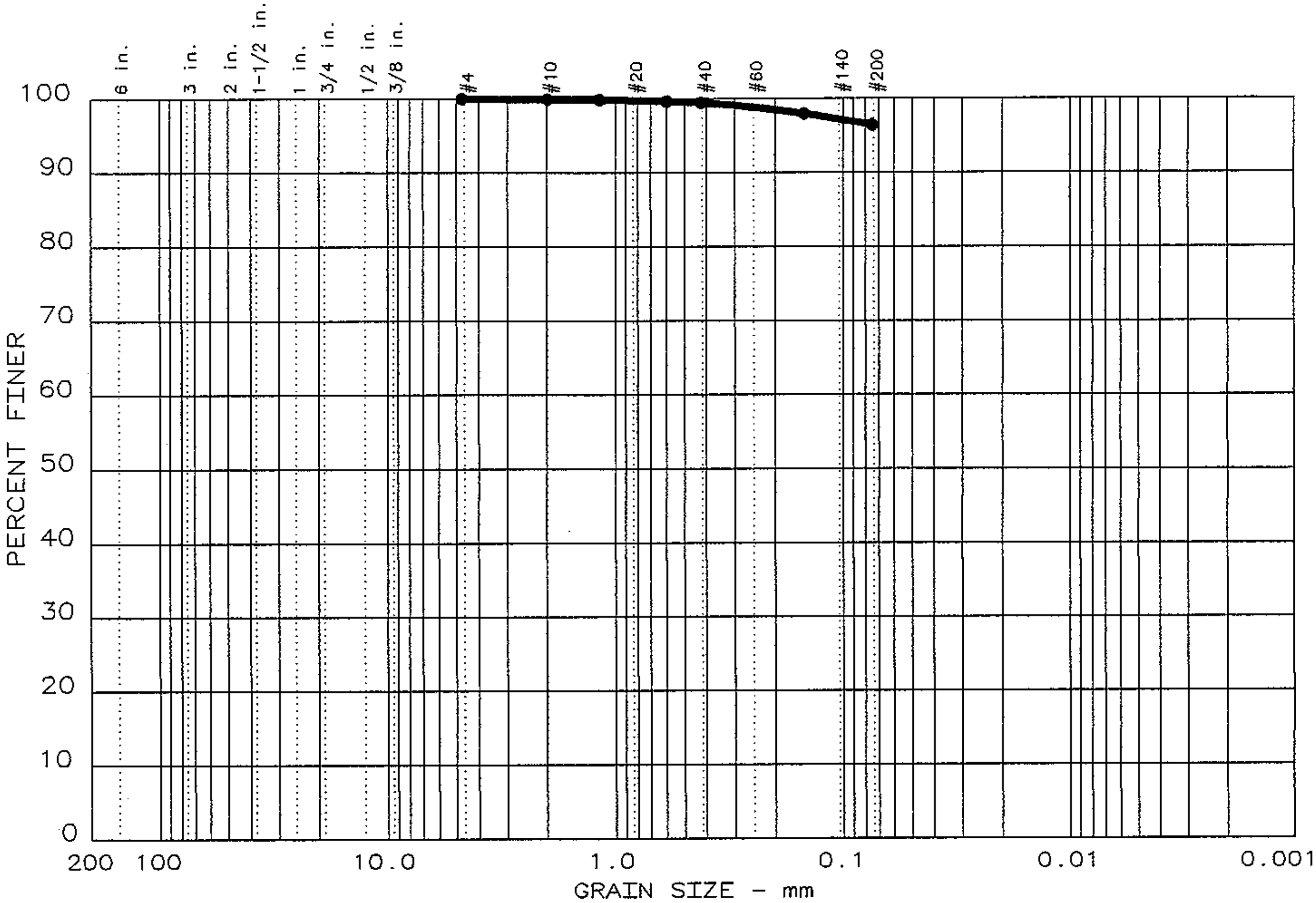
Date: 05.06.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	3.6	96.4	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	48	21								

MATERIAL DESCRIPTION	USCS	AASHTO
● Lean Clay	CL	A-7-6(24)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: P-2 @ 5'

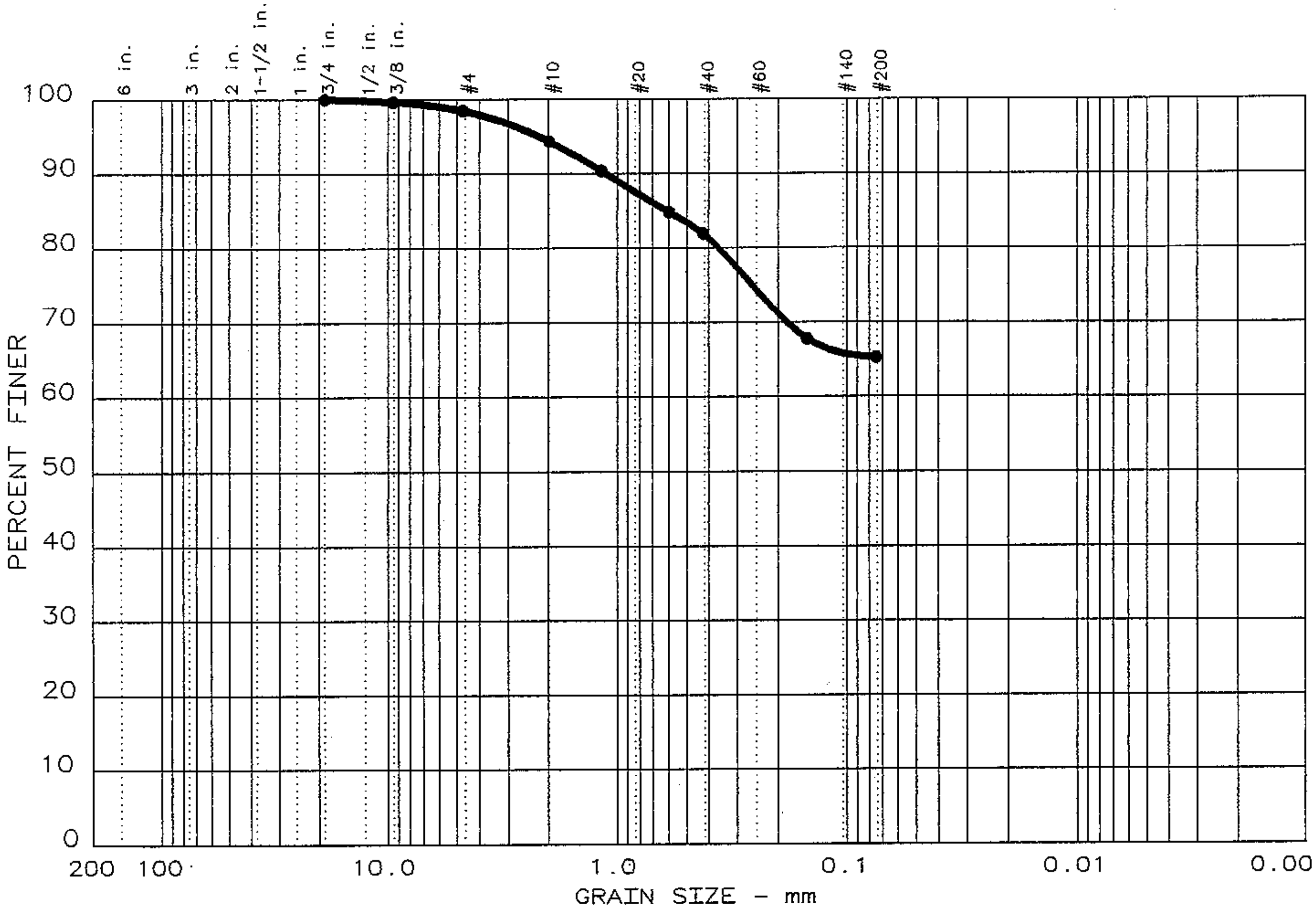
Date: 08.24.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	1.5	33.2	65.3	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	33	15	0.610							

MATERIAL DESCRIPTION	USCS	AASHTO
● Sandy Lean Clay	CL	A-6(8)

Project No.: 65569.01
Project: 4th Street Bridge - Pueblo
● Location: P3 @ 1'-5'

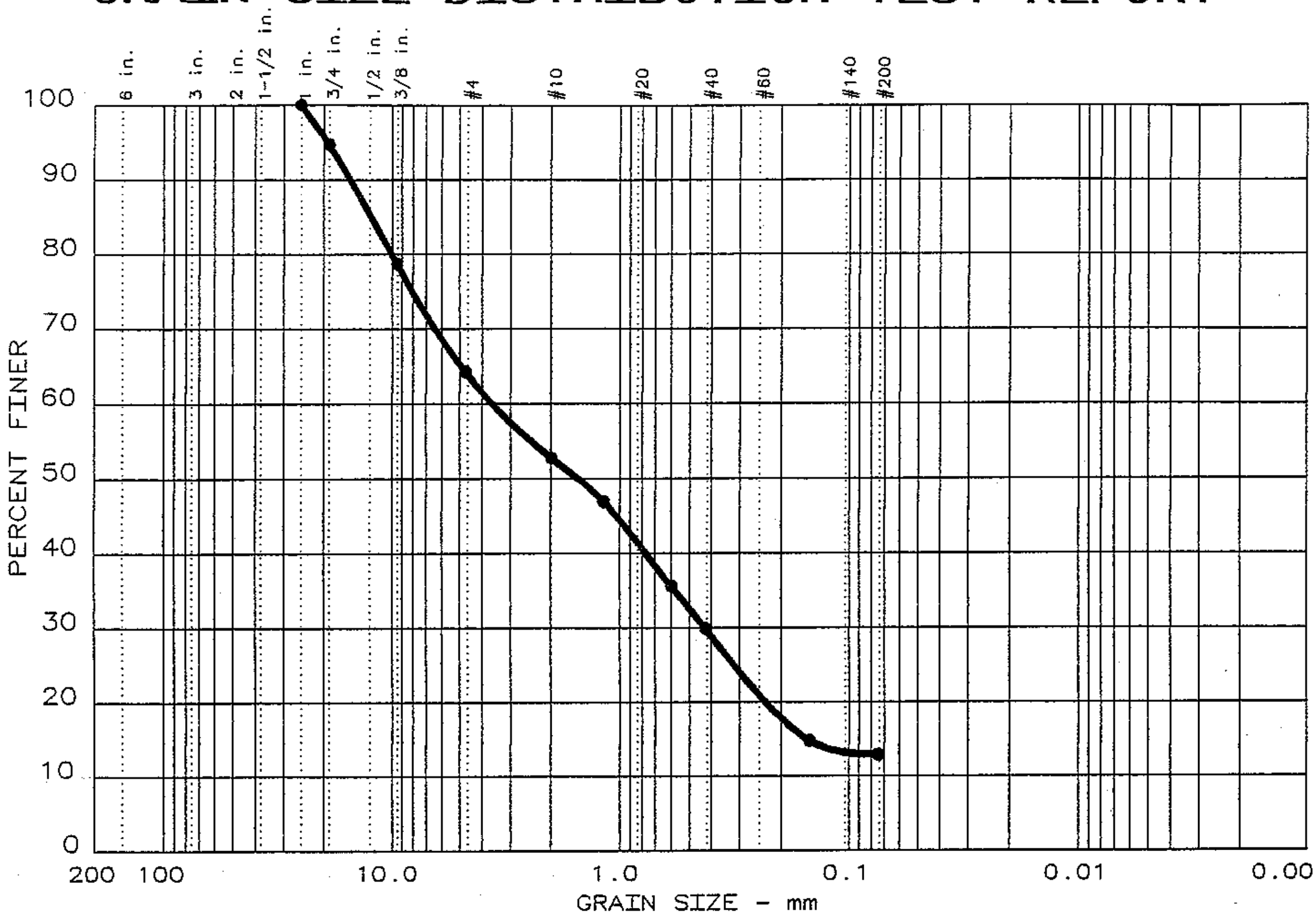
Date: 05.06.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	35.7	51.4	12.9	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	12.3	3.59	1.53	0.427	0.153			

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Sand with Gravel	SM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge
● Location: P4 @ 1'-5'

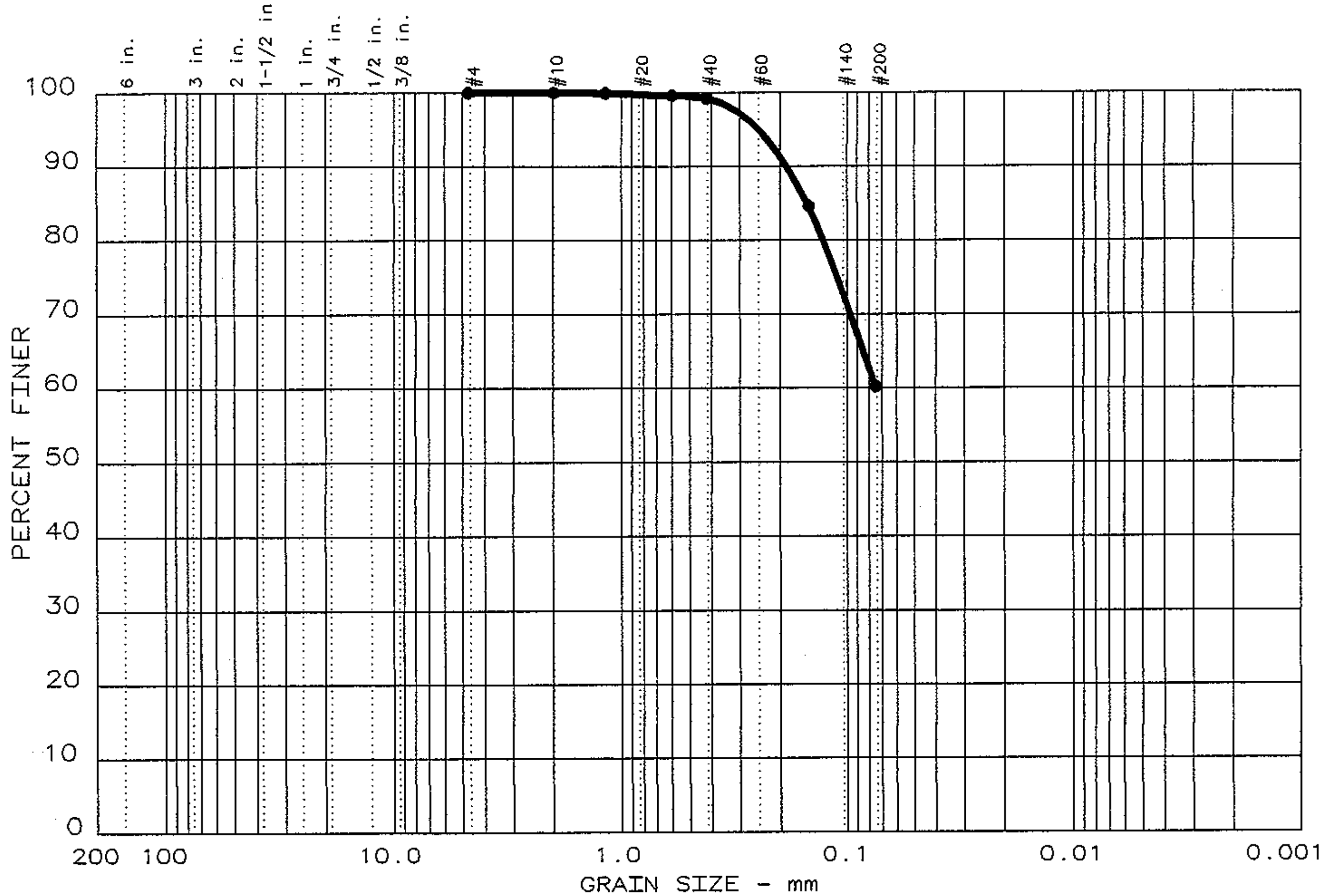
Date: 06.20.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	0.0	39.8	60.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	28	7	0.151							

MATERIAL DESCRIPTION	USCS	AASHTO
● Sandy Silty Clay	CL-ML	A-4(2)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: P-4 @ 5'

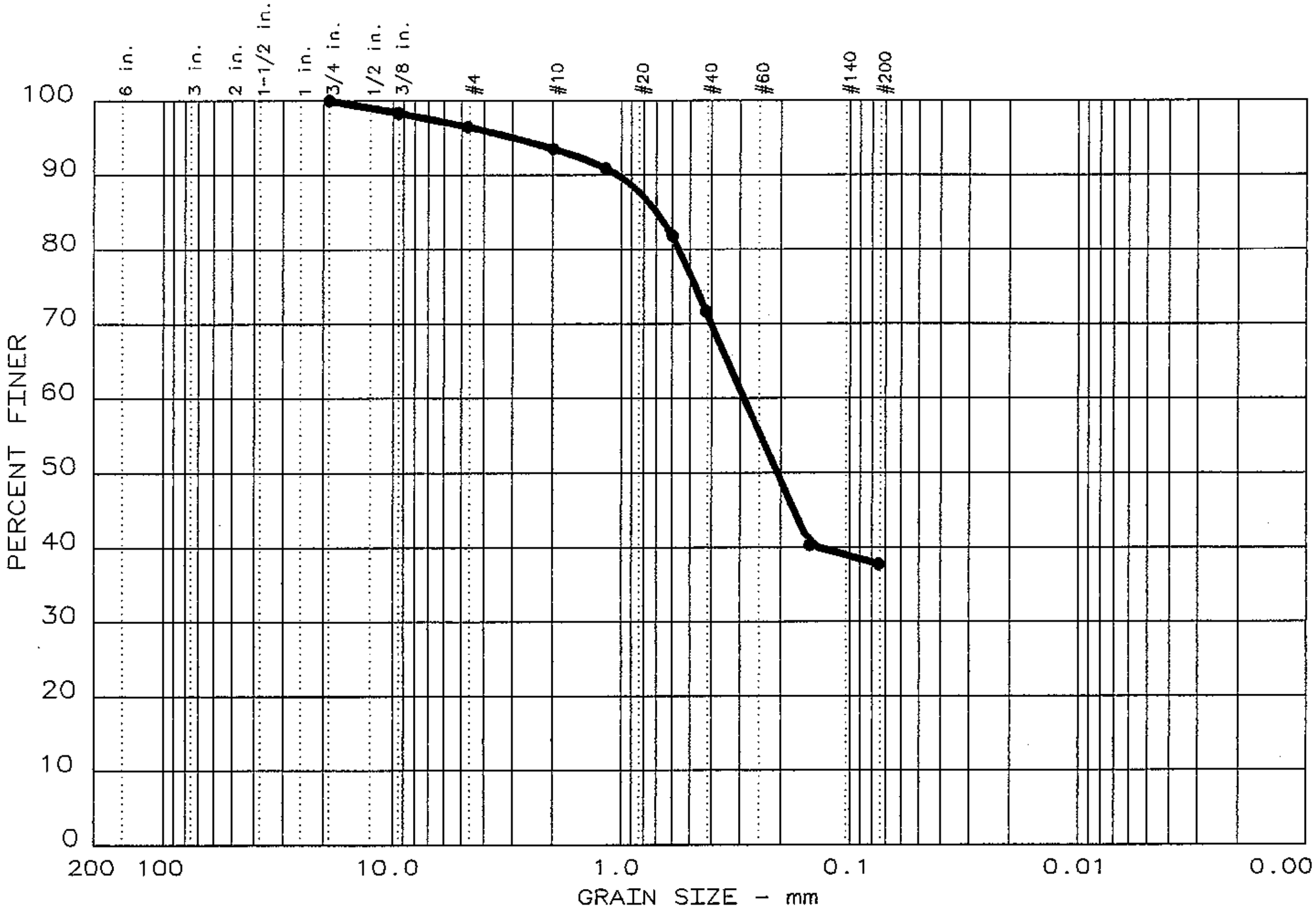
Date: 08.24.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	3.6	58.7	37.7	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	24	4	0.692	0.288	0.207					

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Clayey Sand	SC-SM	A-4(0)

Project No.: 65569.01
Project: 4th Street Bridge - Pueblo
● Location: P5 @ 1'-5'

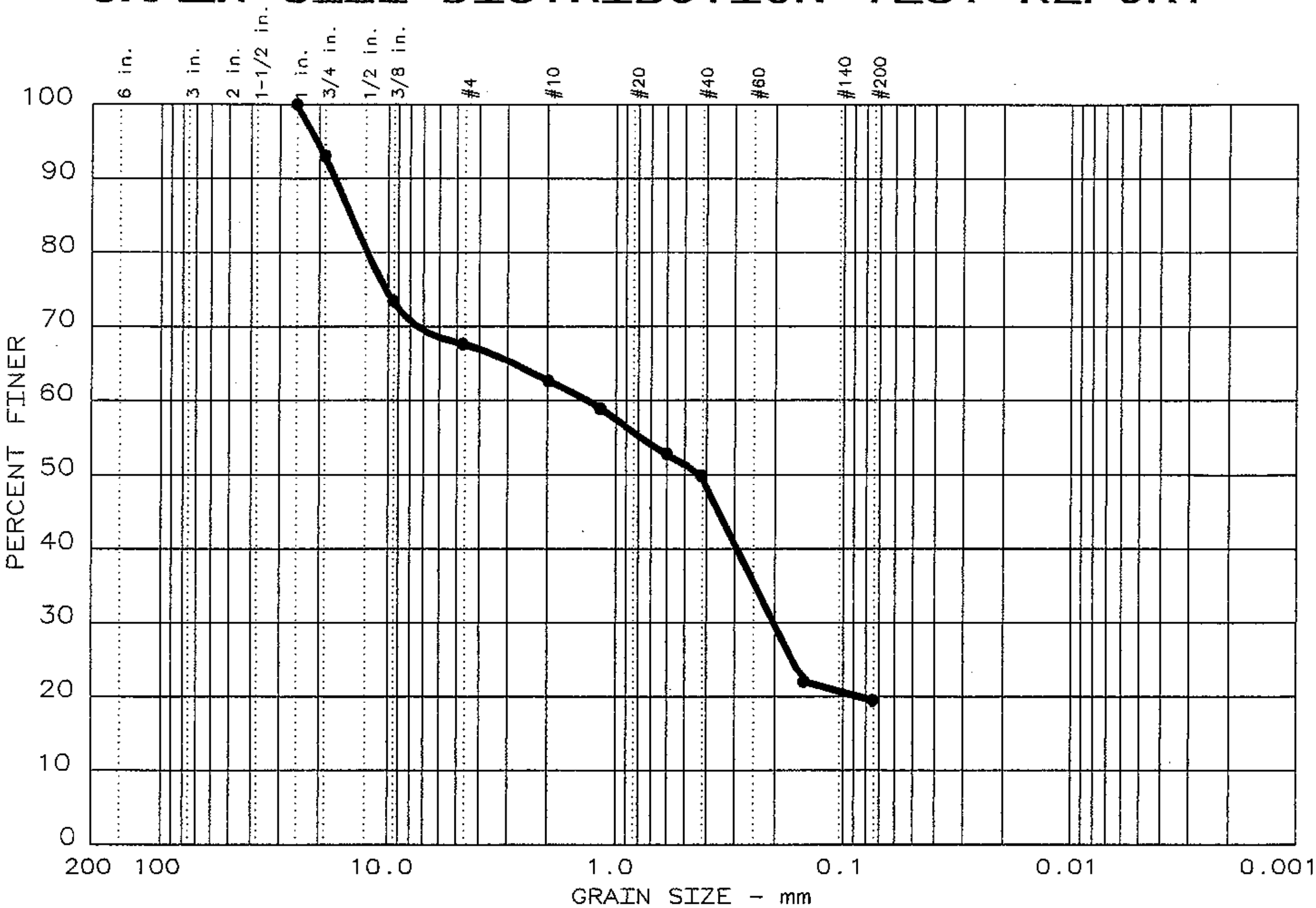
Date: 05.16.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	32.4	48.1	19.5	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	14.6	1.34	0.430	0.201				

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Sand with Gravel	SM	A-1-b

Project No.: 65569.01
Project: 4th Street Bridge - Pueblo
● Location: P6 @ 1'-5'

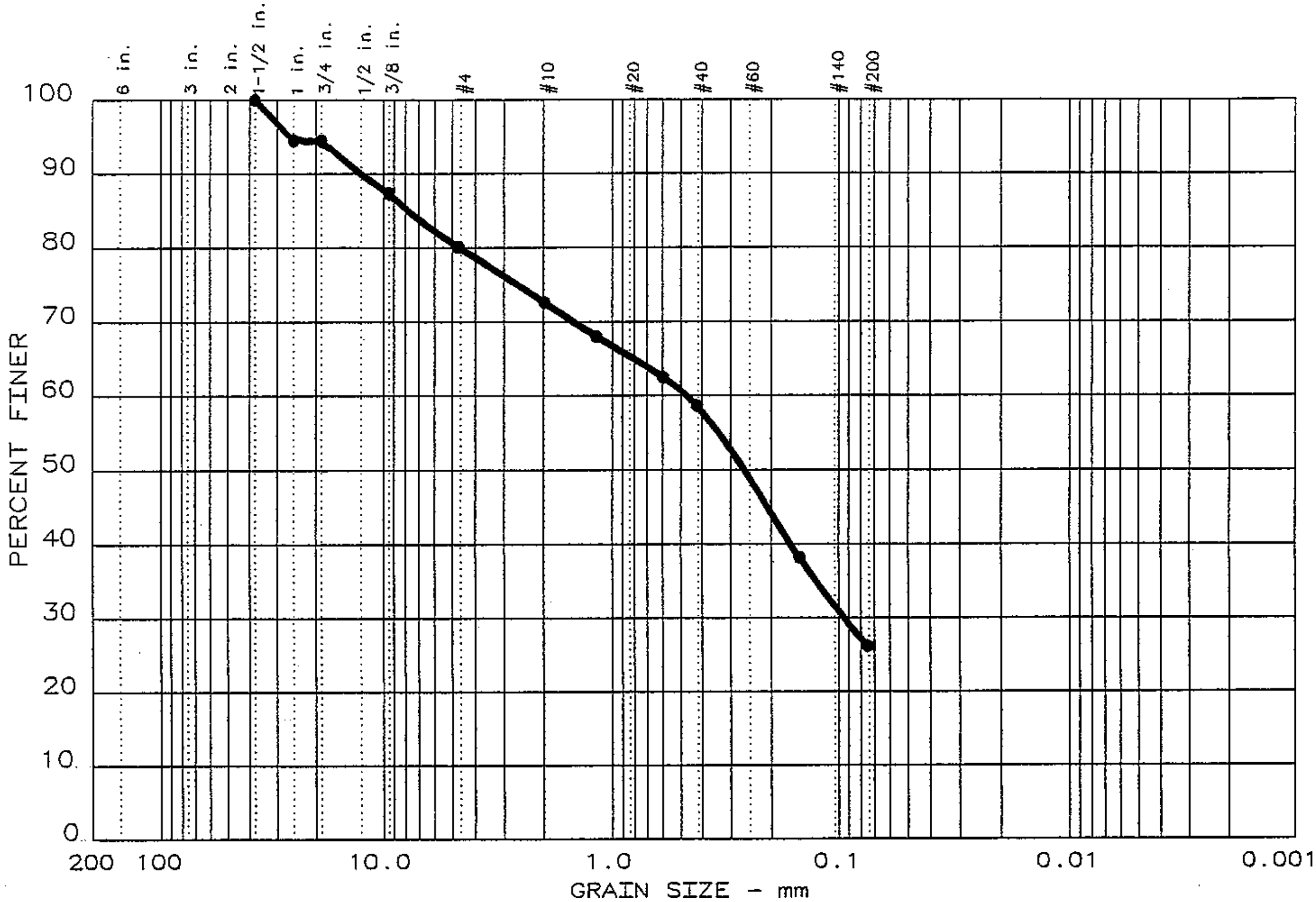
Date: 05.16.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	19.9	53.9	26.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	NV	NP	7.85	0.468	0.263	0.0943				

MATERIAL DESCRIPTION	USCS	AASHTO
● Silty Sand with Gravel	SM	A-2-4(0)

Project No.: 66569.01
Project: 4th Street Bridge
● Location: ET-2 @ 0'-7'

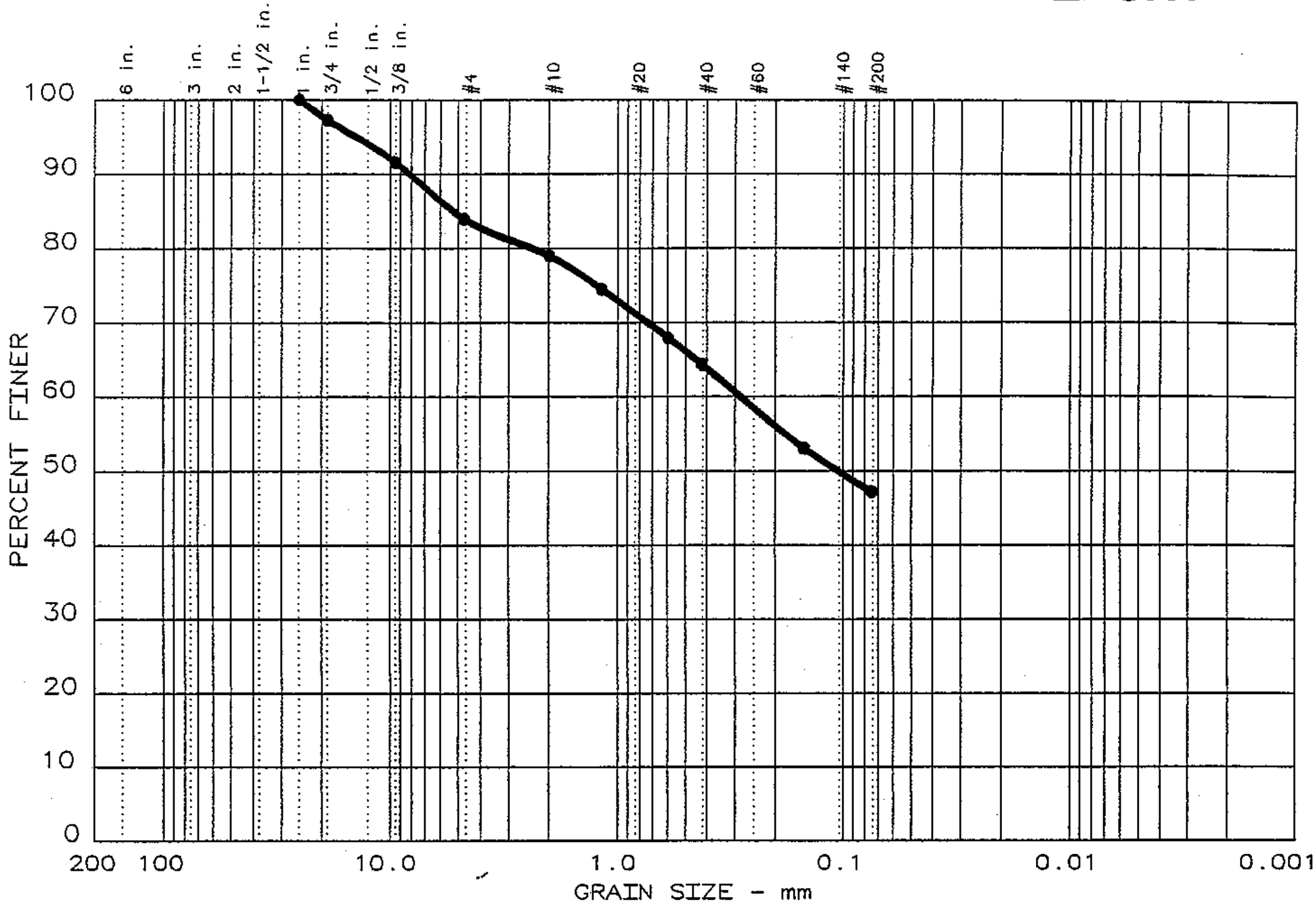
Date: 08.03.06

Remarks:

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Figure No. ______

GRAIN SIZE DISTRIBUTION TEST REPORT



	% +3"	% GRAVEL	% SAND	% SILT	% CLAY
●	0.0	16.0	36.8	47.2	

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●	32	12	5.25	0.285	0.106					

MATERIAL DESCRIPTION	USCS	AASHTO
● Clayey Sand with Gravel	SC	A-6(3)

Project No.: 65569.01
Project: 4th Street Bridge
● Location: ET-2 @ 9'-11'

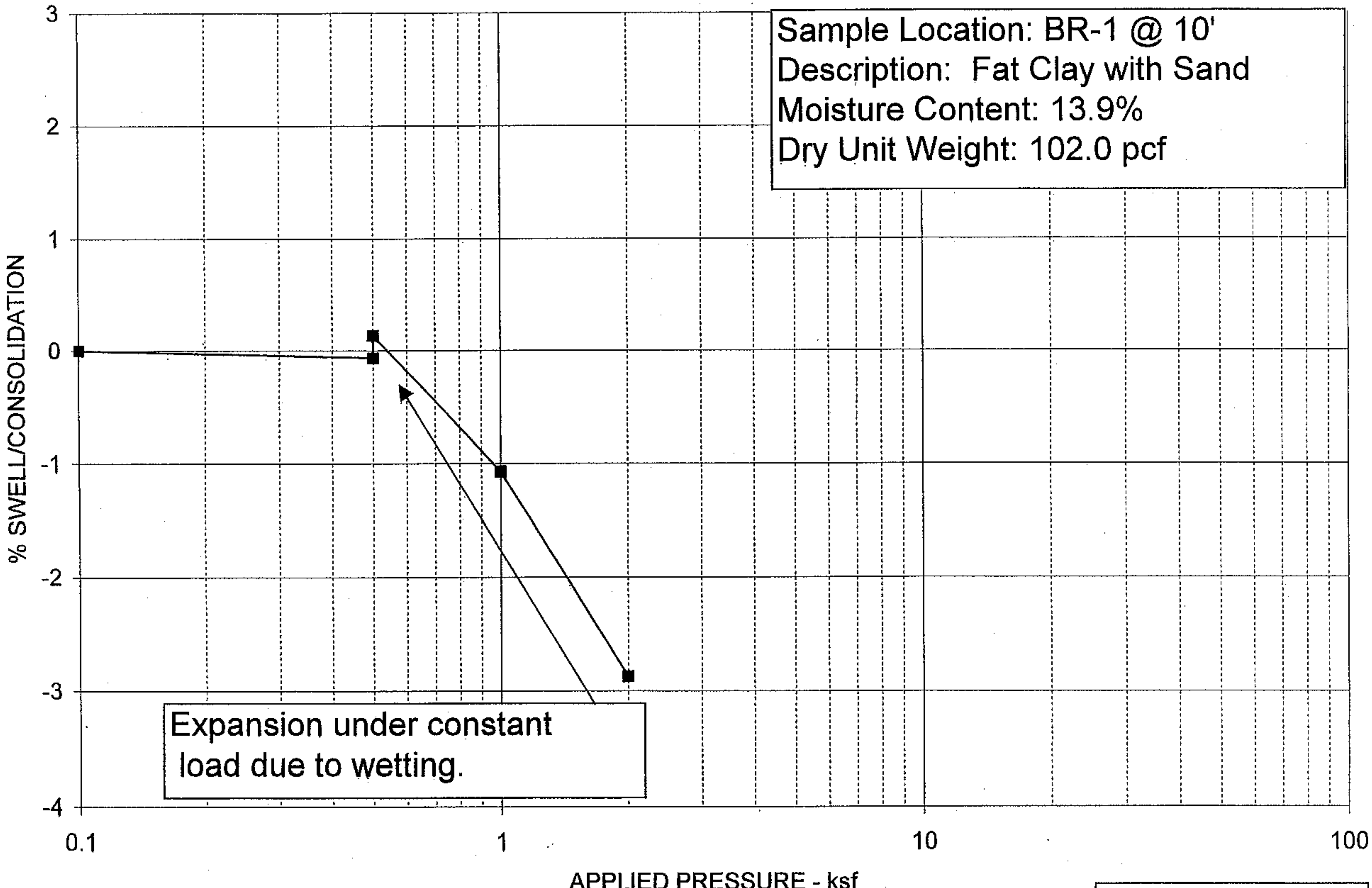
Date: 08.03.06

GRAIN SIZE DISTRIBUTION TEST REPORT
GOODSON & ASSOCIATES, INC.
Consulting Engineers

Remarks:

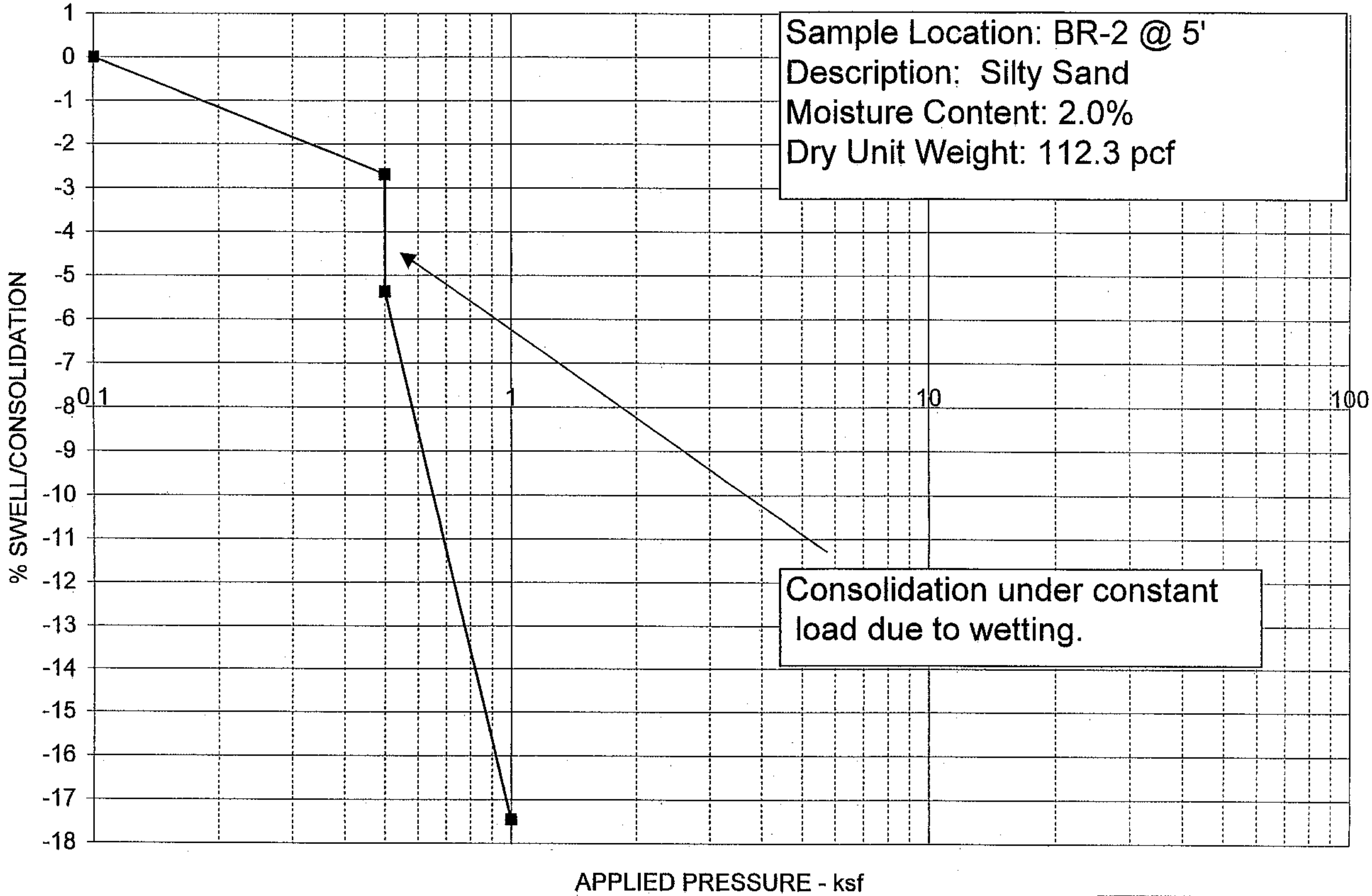
Figure No. ______

GOODSON & ASSOCIATES, INC.
SWELL-CONSOLIDATION TEST



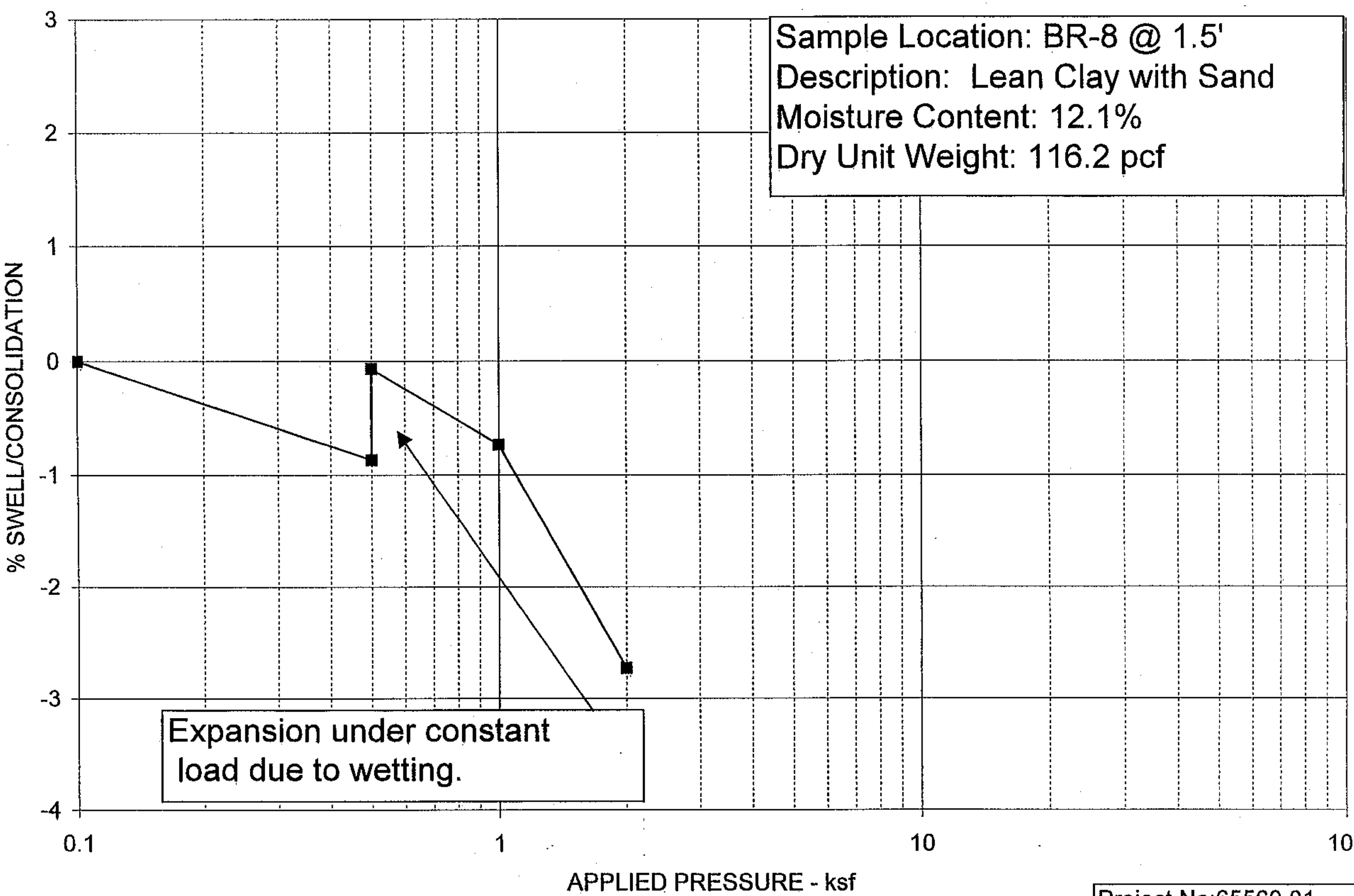
Project No:65569.01

GOODSON & ASSOCIATES, INC.
SWELL-CONSOLIDATION TEST

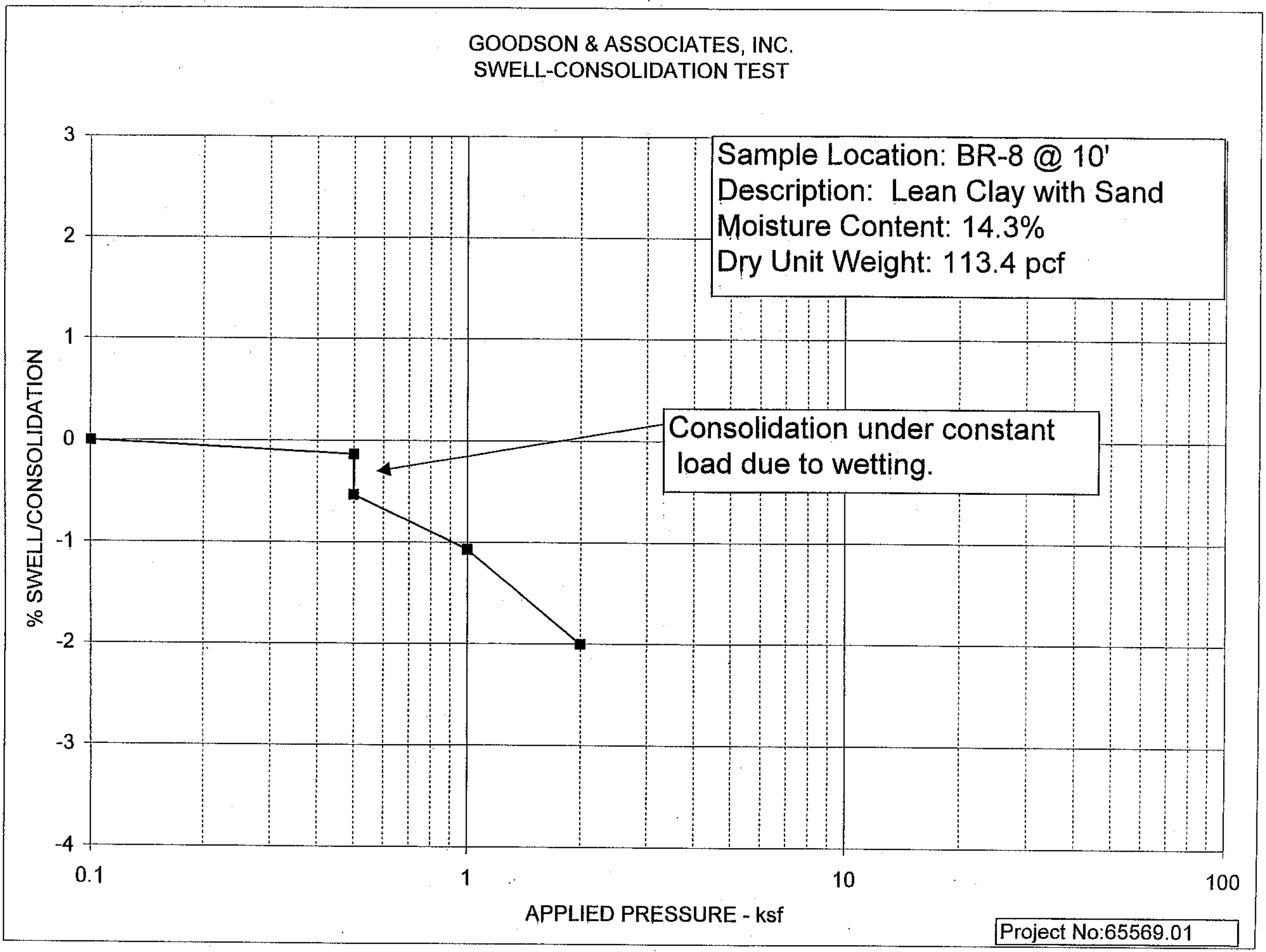


Project No:65569.01

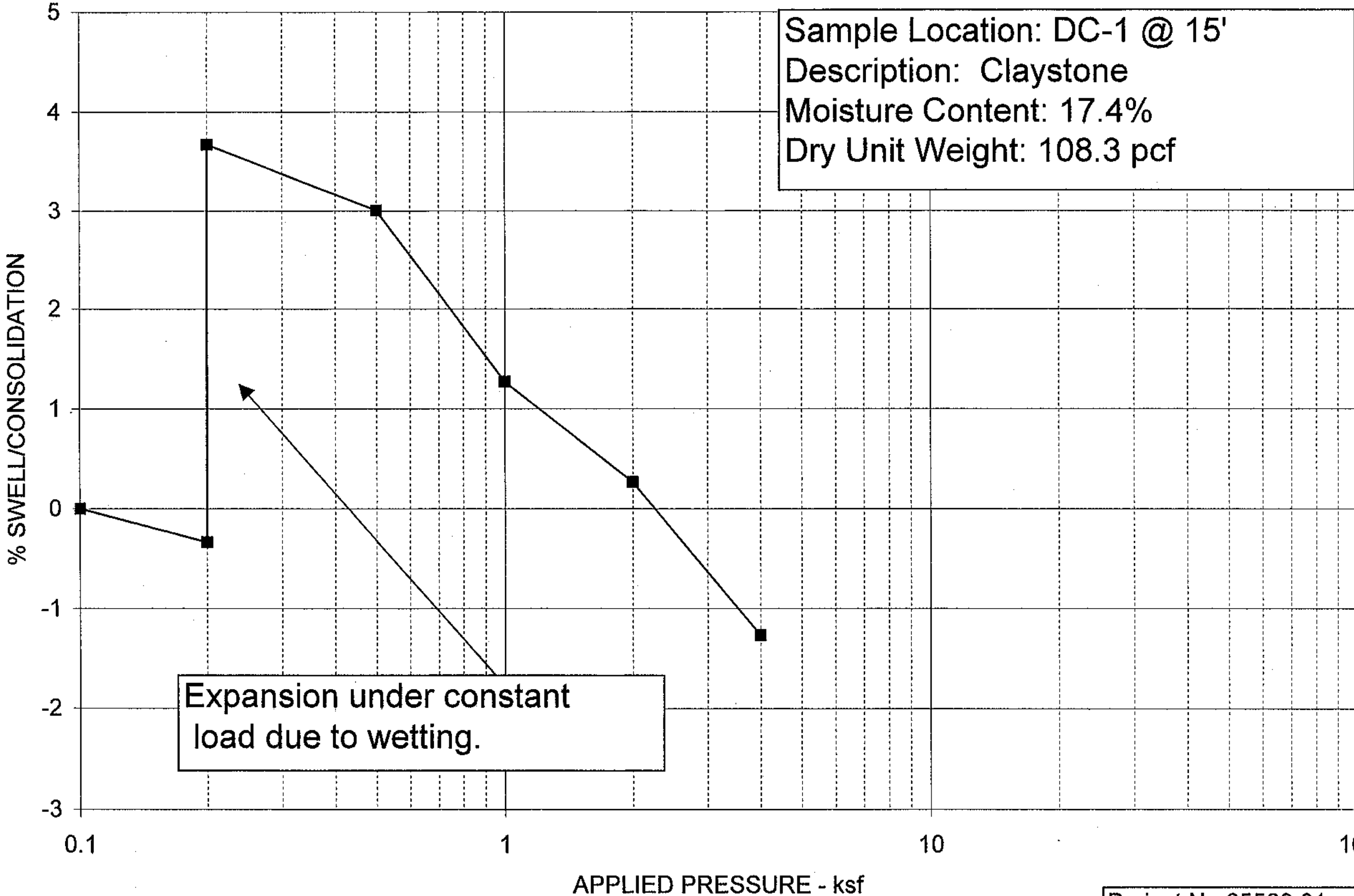
GOODSON & ASSOCIATES, INC.
SWELL-CONSOLIDATION TEST



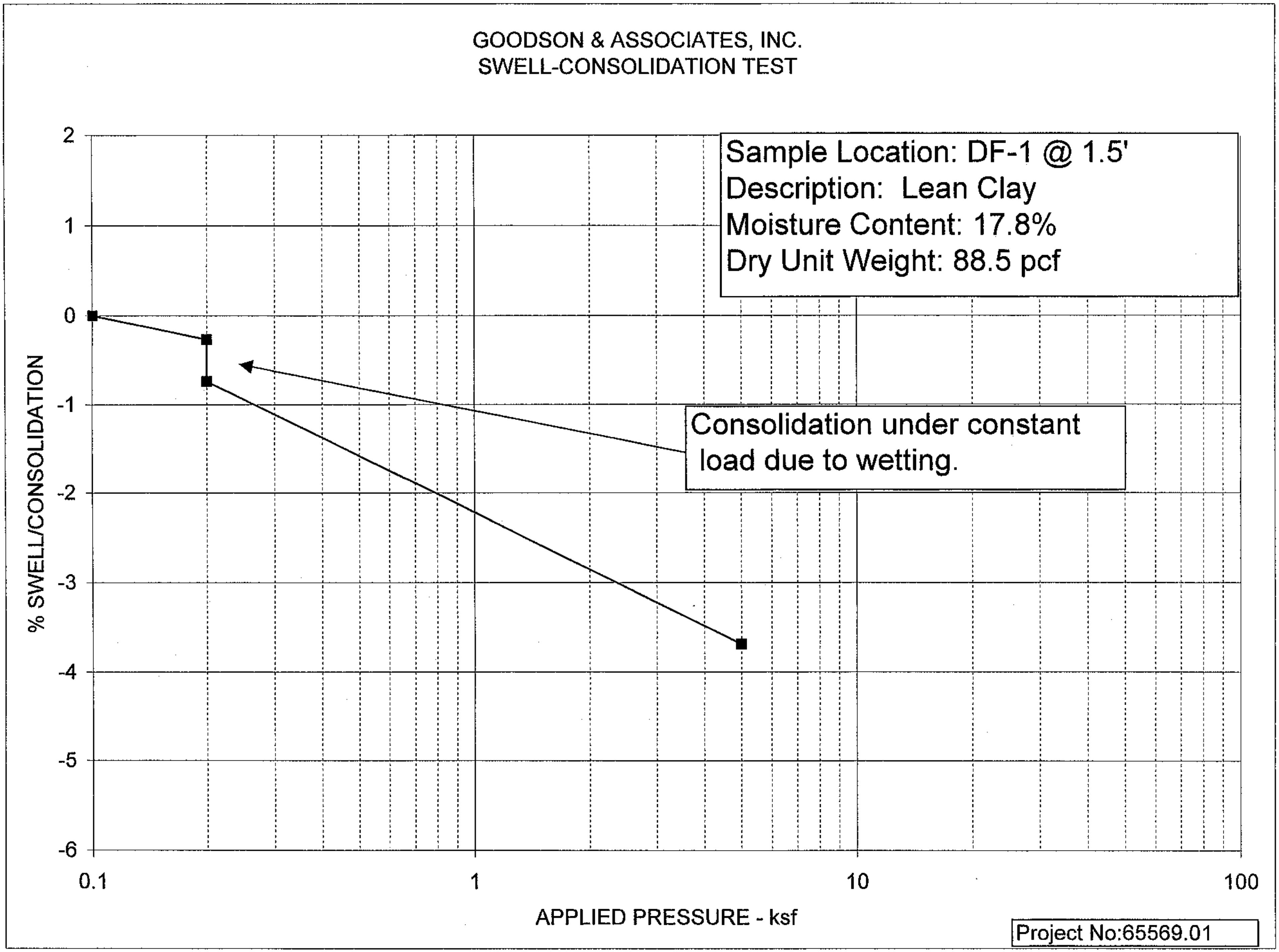
Project No:65569.01


GOODSON & ASSOCIATES, INC.
SWELL-CONSOLIDATION TEST
Sample Location: BR-8 @ 10'
Description: Lean Clay with Sand
Moisture Content: 14.3%
Dry Unit Weight: 113.4 pcf
Consolidation under constant load due to wetting.
% SWELL/CONSOLIDATION
3
2
1
0
-1
-2
-3
-4
0.1
1
10
100
APPLIED PRESSURE - ksf
Project No:65569.01

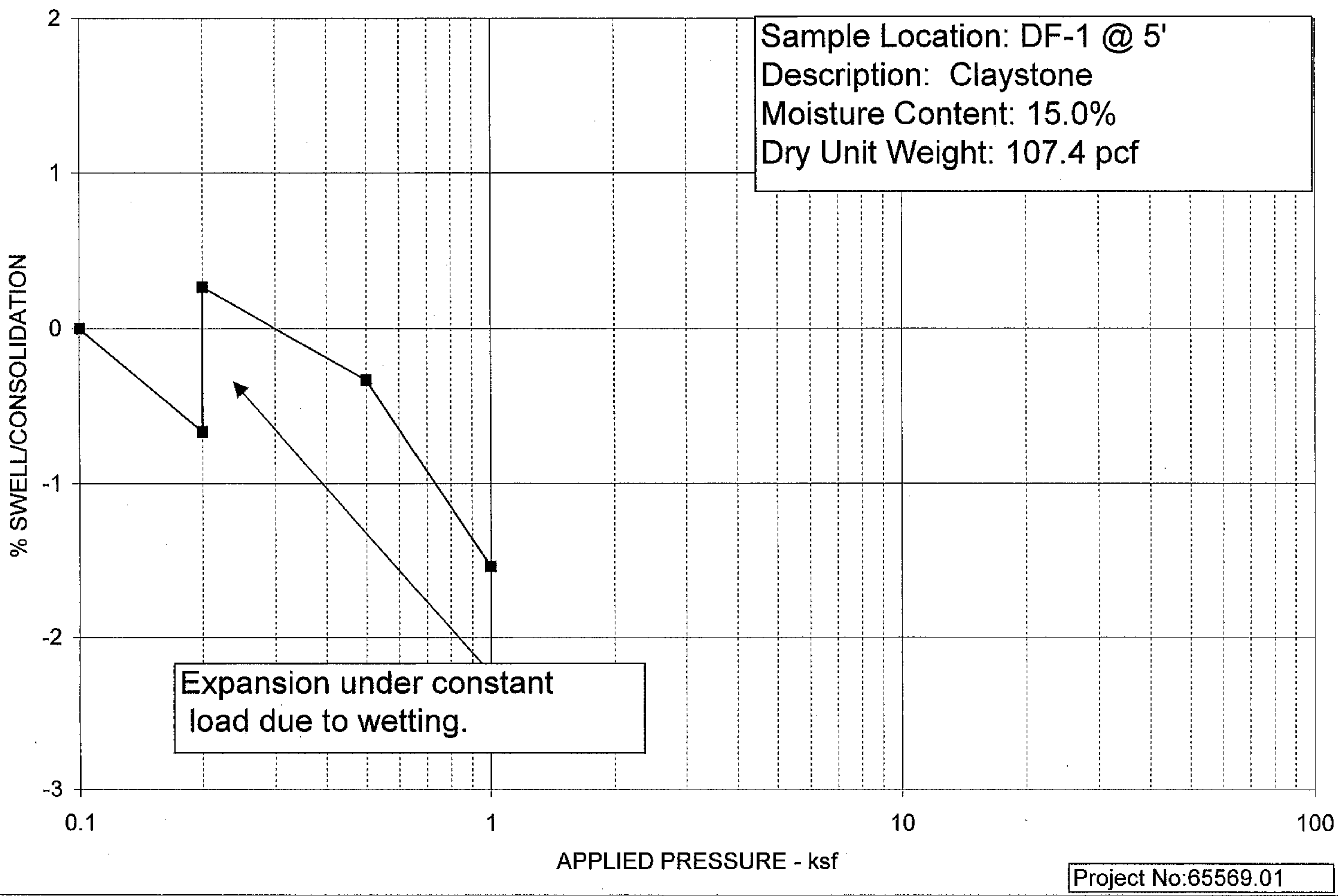
GOODSON & ASSOCIATES, INC.
SWELL-CONSOLIDATION TEST



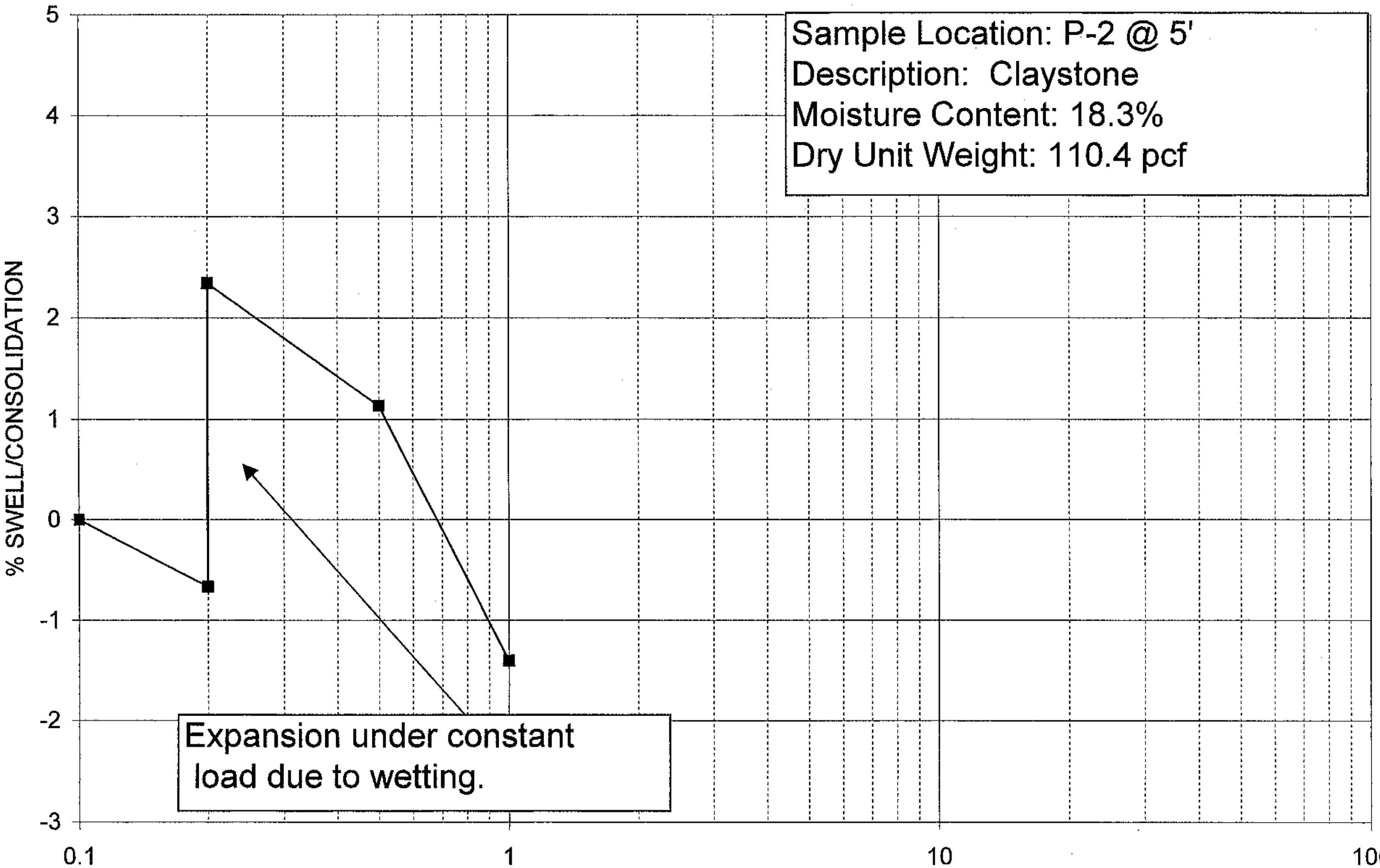
Project No:65569.01


GOODSON & ASSOCIATES, INC.
SWELL-CONSOLIDATION TEST
Sample Location: DF-1 @ 1.5'
Description: Lean Clay
Moisture Content: 17.8%
Dry Unit Weight: 88.5 pcf
Consolidation under constant
load due to wetting.
% SWELL/CONSOLIDATION
2
1
0
-1
-2
-3
-4
-5
-6
0.1
1
10
100
APPLIED PRESSURE - ksf
Project No:65569.01

GOODSON & ASSOCIATES, INC.
SWELL-CONSOLIDATION TEST



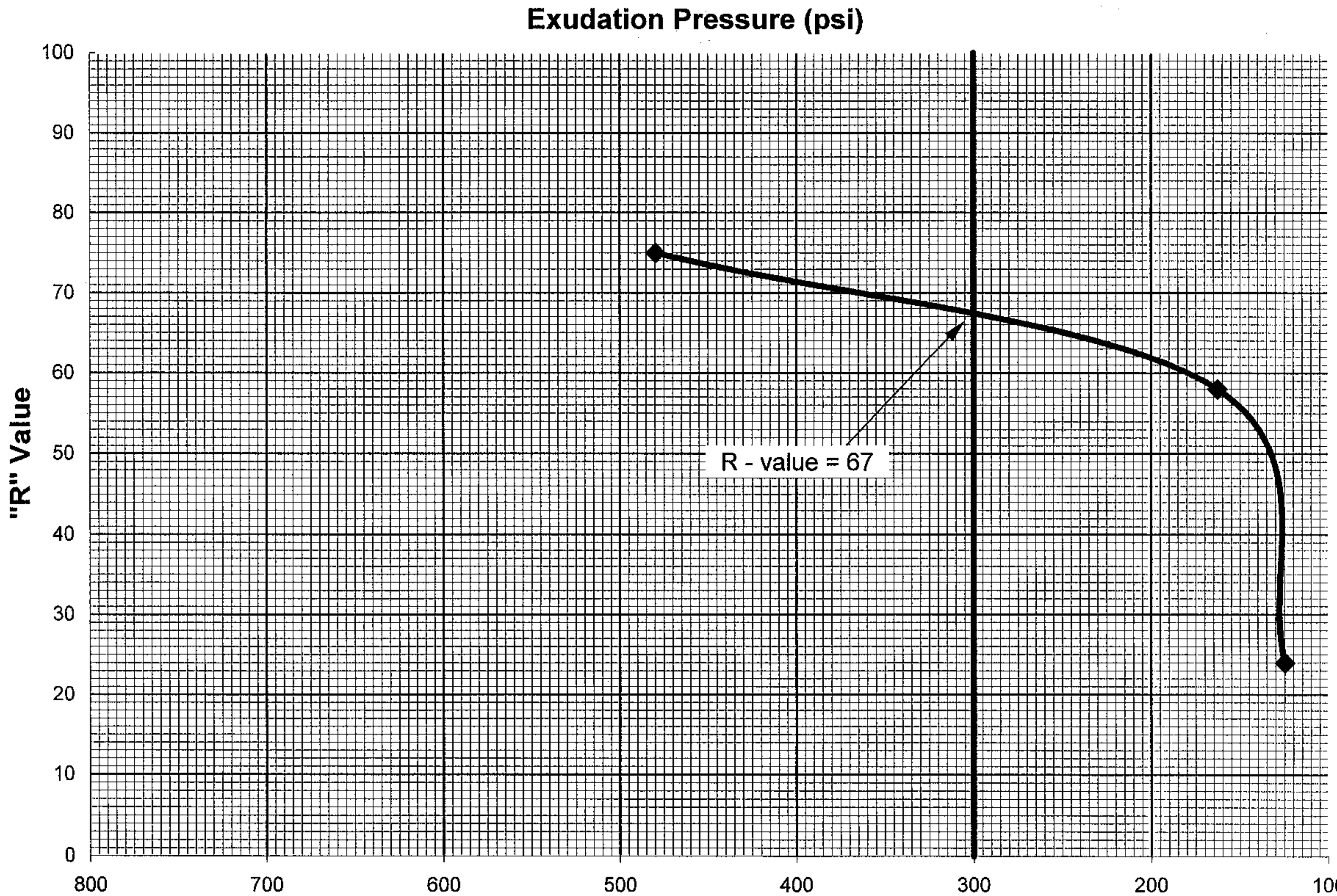
GOODSON & ASSOCIATES, INC.
SWELL-CONSOLIDATION TEST



Project No:65569.01

Goodson & Associates, Inc.
Stabilometer Graph

65569.01
4th Street Bridge
P1 @ 1'-5'

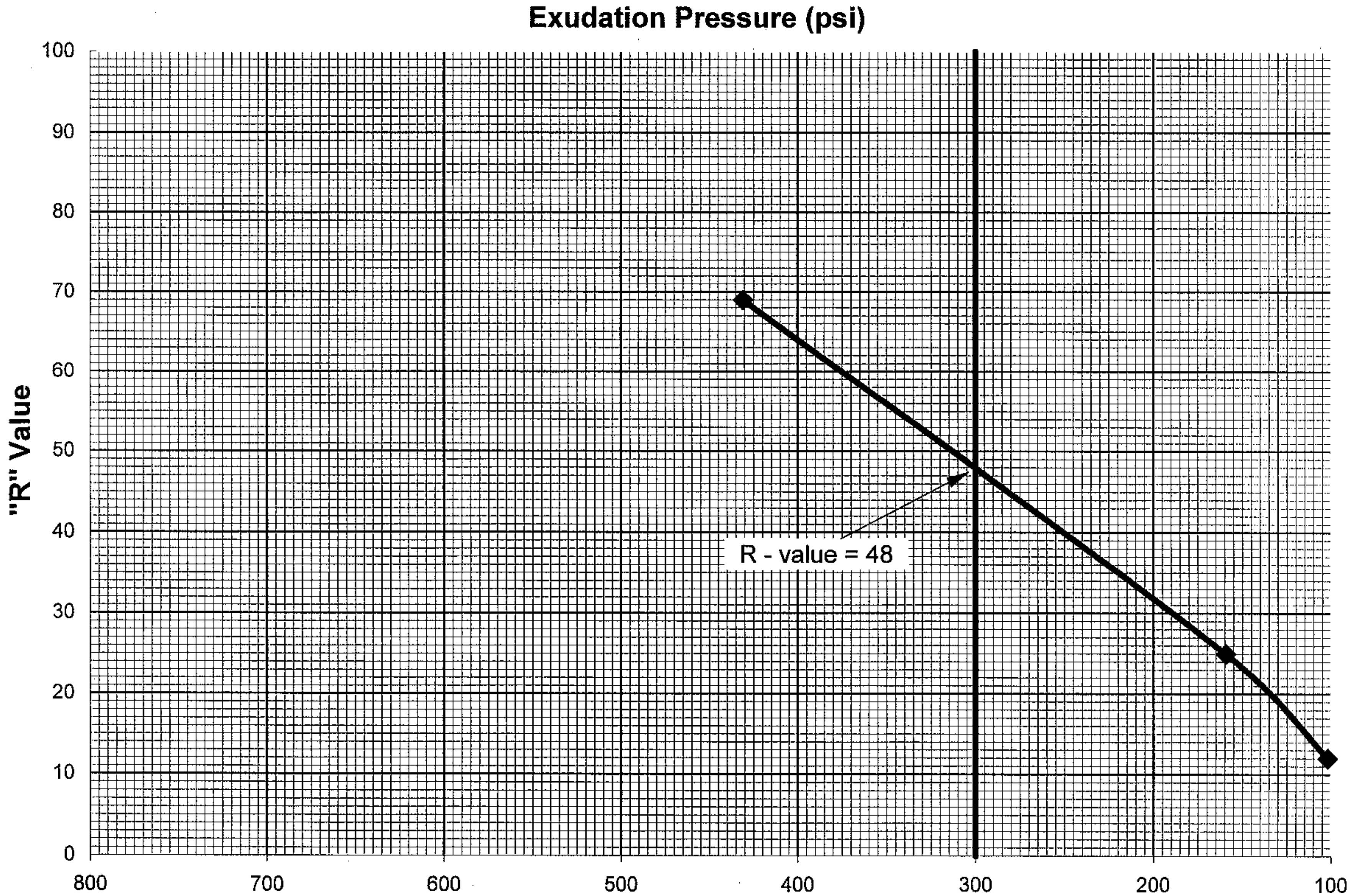


Date:06.20.2006

CDOT Form #1003 1/99

Goodson & Associates, Inc.
Stabilometer Graph

65569.01
4th Street Bridge
P2 @ 0'-5'



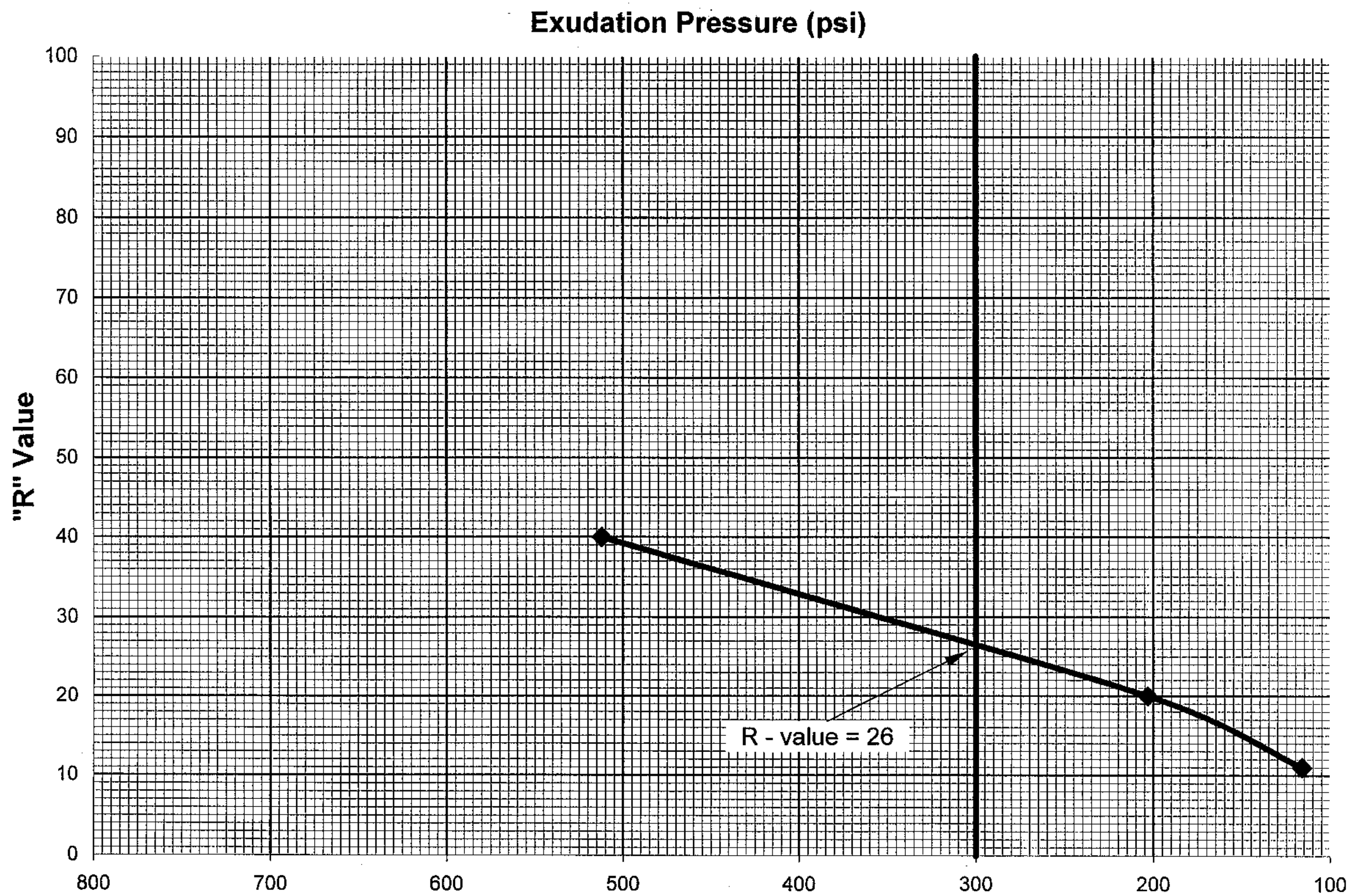
Date:06.20.2006

CDOT Form #1003 1/99

Goodson & Associates, Inc.

Stabilometer Graph

65569.01
4th Street Bridge
P3 @ 1'-5'

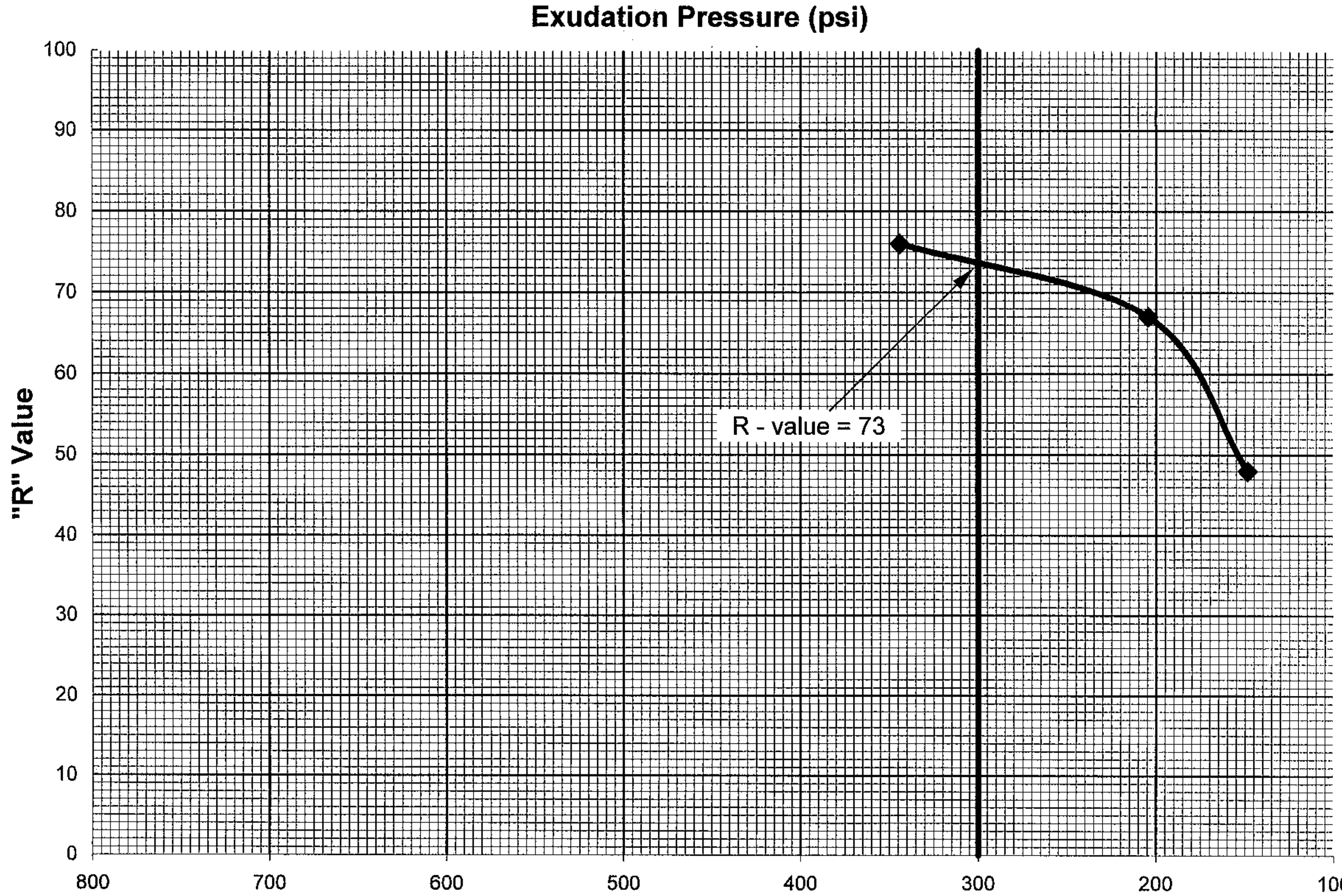


Date:06.20.2006

CDOT Form #1003 1/99

Goodson & Associates, Inc.
Stabilometer Graph

65569.01
4th Street Bridge
P5 @ 1'-5'

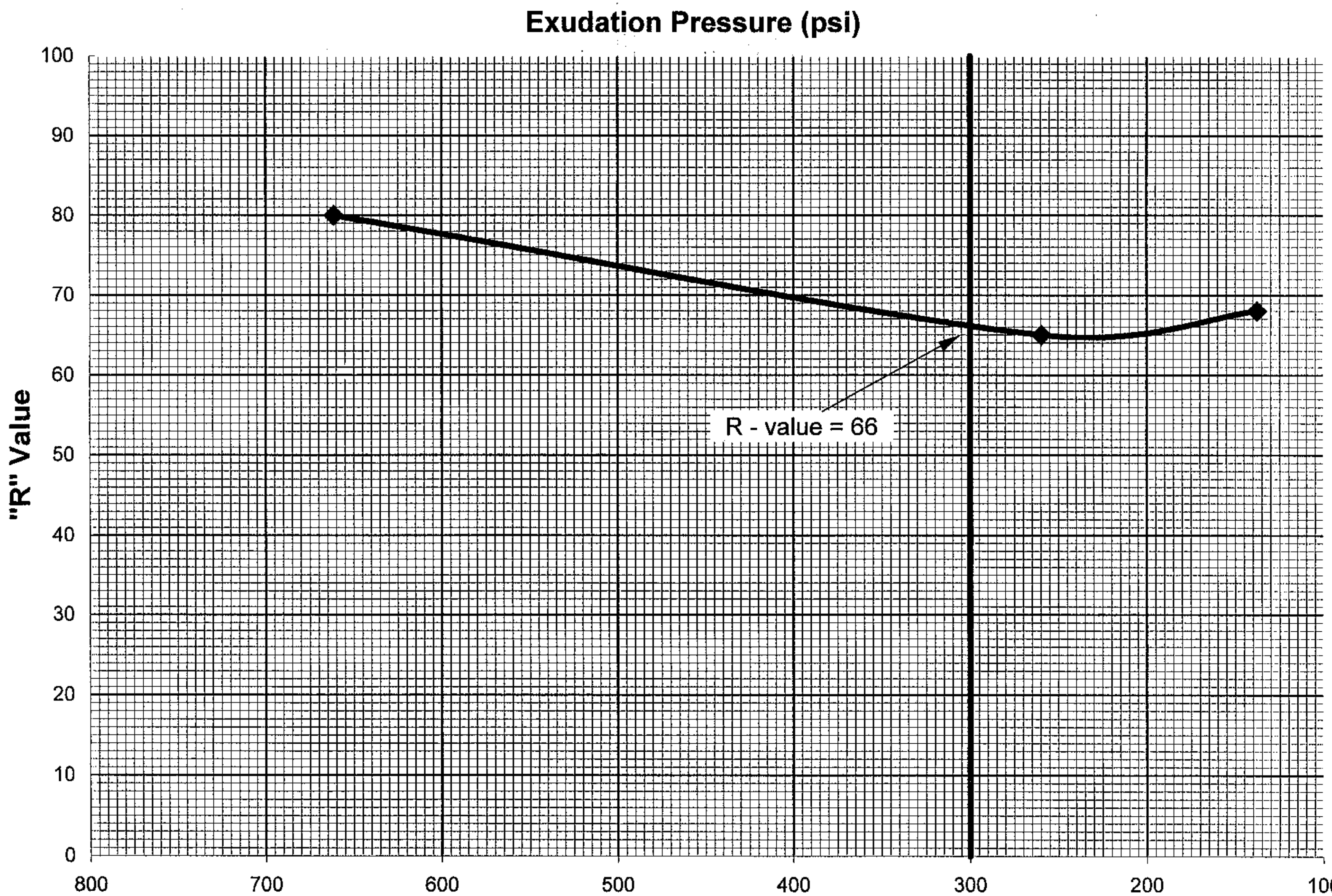


Date:06.20.2006

CDOT Form #1003 1/99

Goodson & Associates, Inc.
Stabilometer Graph

65569.01
4th Street Bridge
P6 @ 1'-5'

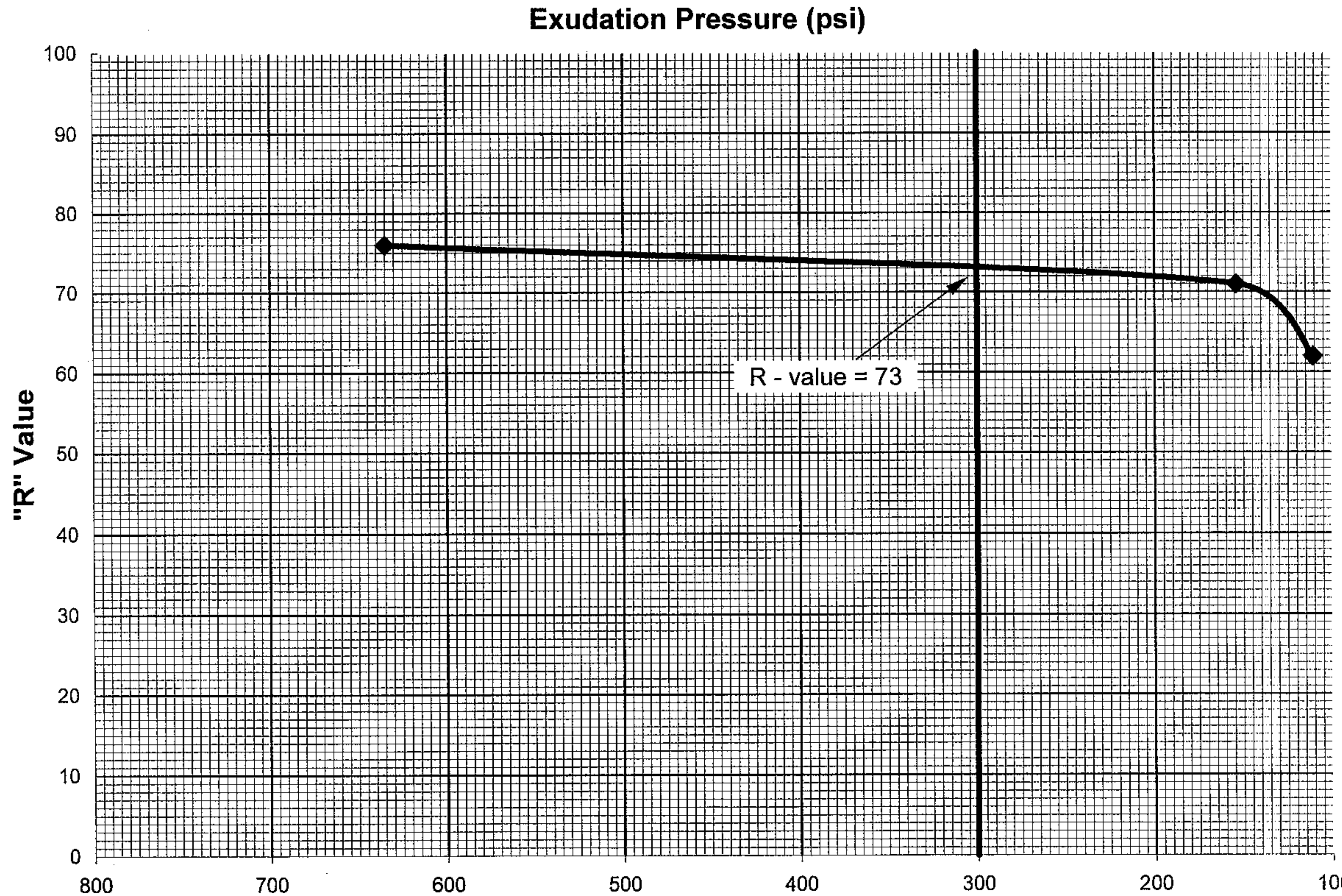


Date:06.20.2006

CDOT Form #1003 1/99

Goodson & Associates, Inc.
Stabilometer Graph

65569.01
4th Street Bridge
BR-5 @ 1'-5'



Date:09.26.2006

CDOT Form #1003 1/99

UNCONFINED COMPRESSIVE STRENGTH
With Stress / Strain Measurements
ASTM D 3148

CLIENT: Goodson & associates, Inc.

JOB NO.: 2014-104

LOCATION: 4th Street Bridge

DATE TESTED: 7/3/06 HN

Specimen ID			Diameter (in.)	Length (in.)	Mass (gms)	Wet Density (pcf)	Failure Load (lb)	Failure Type **	Compressive Strength (psi)	Young's Modulus (X10^6 psi)	Poisson's Ratio
Boring	Depth (ft.)	Rock type									
BR-1	41.0-42.0		2.407	5.228	947.20	151.7	40,250	S/F	8,850	1.21	0.083
BR-1	47.0-48.5		2.410	5.348	942.10	147.1	11,250	S	2,470	0.99	0.064
BR-2	30.2-32.0		2.403	5.247	918.30	147.0	49,250	C	10,860	1.38	0.122
BR-2	45.7-47.2		2.408	5.402	939.90	145.5	7,625	S	1,670	0.29	0.041
BR-3	11.1-11.8		2.401	3.943	657.40	140.3	5,800	S	*1250	0.12	0.041
BR-3	24.0-25.0		2.405	4.963	890.10	150.4	4,750	F	1,050	0.15	0.024

Notes and Comments:

* Indicates L/D < 2.0. Correction Factor for short sample was applied..

C=Ca/[0.88+0.24b/h]

Ca = Failure Load / Surface Area

b = Sample Diameter

h = Sample Length

** Failure Type:

S: Shear Failure, M: Matrix Failure, F/V Fracture, Bedding/Void Collapse, C: Combination

Data Entered By: HN Date: 07/03/2006

Data Checked By: CJ Date: 07 05/06

Filename: GDUCSSR1

ADVANCED TERRA TESTING, Inc.

UNCONFINED COMPRESSIVE STRENGTH
With Stress / Strain Measurements
ASTM D 7012; Method D (Previously ASTM D 3148)

CLIENT: Goodson & Associates, Inc.

JOB NO.: 2014-105

LOCATION: SH 96, 4th Street Bridge Site

DATE TESTED: 8/28-31/06 HN

Specimen ID			Diameter (in.)	Length (in.)	Mass (gms)	Wet Density (pcf)	Failure Load (lb)	Failure Type *	Compressive Strength (psi)	Young's Modulus (X10^6 psi)	Poisson's Ratio
Boring	Depth (ft.)	Rock type									
BR-1	65.5-66.4		2.410	5.434	964.10	148.2	11,600	F/S	2,540	0.56	0.079
BR-1	100.0-101.9		2.401	4.995	890.10	149.9	17,600	F	3,890	1.58	0.087
BR-2	55.6-57.3		2.410	5.391	917.30	142.1	11,800	S	2,590	0.33	0.067
BR-2	69.8-71.4		2.402	5.159	910.10	148.3	57,000	C	12,580	2.26	0.229
BR-3	32.8-33.6		2.398	5.066	870.10	144.9	10,600	S	2,350	0.39	0.085
BR-6	28.5-30.5		2.401	5.308	920.10	145.9	16,800	S	3,710	0.55	0.071
BR-6	40.8-41.8		2.397	5.220	923.10	149.3	37,200	S/F	8,240	1.32	0.161
BR-7	45.0-45.5		2.399	5.248	906.10	145.5	44,400	F/S	9,820	1.17	0.164
BR-7	55.0-55.6		2.358	4.984	834.10	146.0	29,600	S	6,780	1.44	0.153
BR-8	48.8-50.0		2.397	5.194	923.80	150.2	21,600	F	4,790	1.08	0.199
BR-8	61.0-62.0		2.411	5.298	921.00	145.1	57,000	F	12,490	1.09	0.139

Notes and Comments: * Failure Type:
S: Shear Failure, M: Matrix Failure, F/V Fracture, Bedding/Void Collapse, C: Combination

Data Entered By: HN Date: 09/05/2006
Data Checked By: KR Date: 9/5/06
Filename: GDUCSSR2

ADVANCED TERRA TESTING, Inc.

UNCONFINED COMPRESSIVE STRENGTH
With Stress / Strain Measurements
ASTM D 7012; Method D (Previously ASTM D 3148)

CLIENT: Goodson & Associates, Inc.

JOB NO.: 2014-106

LOCATION: SH 96, 4th Street Bridge Site

DATE TESTED: 9/27-29/06 HN

Specimen ID			Diameter (in.)	Length (in.)	Mass (gms)	Wet Density (pcf)	Failure Load (lb)	Failure Type **	Compressive Strength (psi)	Young's Modulus (X10^6 psi)	Poisson's Ratio
Boring	Depth (ft.)	Rock type									
BR-5	39.0		2.501	5.297	1018.10	149.0	13,500	F	2,750	4.63	0.247
***BR-5	57.57.8		2.498	5.330	1008.10	147.0	7,500	F	1,530	7.11	0.325
+BR-5	69.2-69.8		2.495	4.323	804.00	144.9	2,200	F	*440	0.13	0.019
BR-5	72.0-72.9		2.499	5.271	1015.60	149.7	23,000	C	4,690	0.83	0.079
BR-4	36.3-37.2		2.494	5.433	1027.90	147.5	14,750	F	3,020	1.32	0.074
BR-4	46.8-48.2		2.500	5.533	1068.40	149.9	31,250	F	6,370	1.78	0.117
BR-4	55.5-57.5		2.499	5.255	1020.50	150.8	14,750	S/F	3,010	0.68	0.128
BR-4	65.0-65.8		2.502	5.517	1040.10	146.1	5,000	F	1,020	0.66	0.121

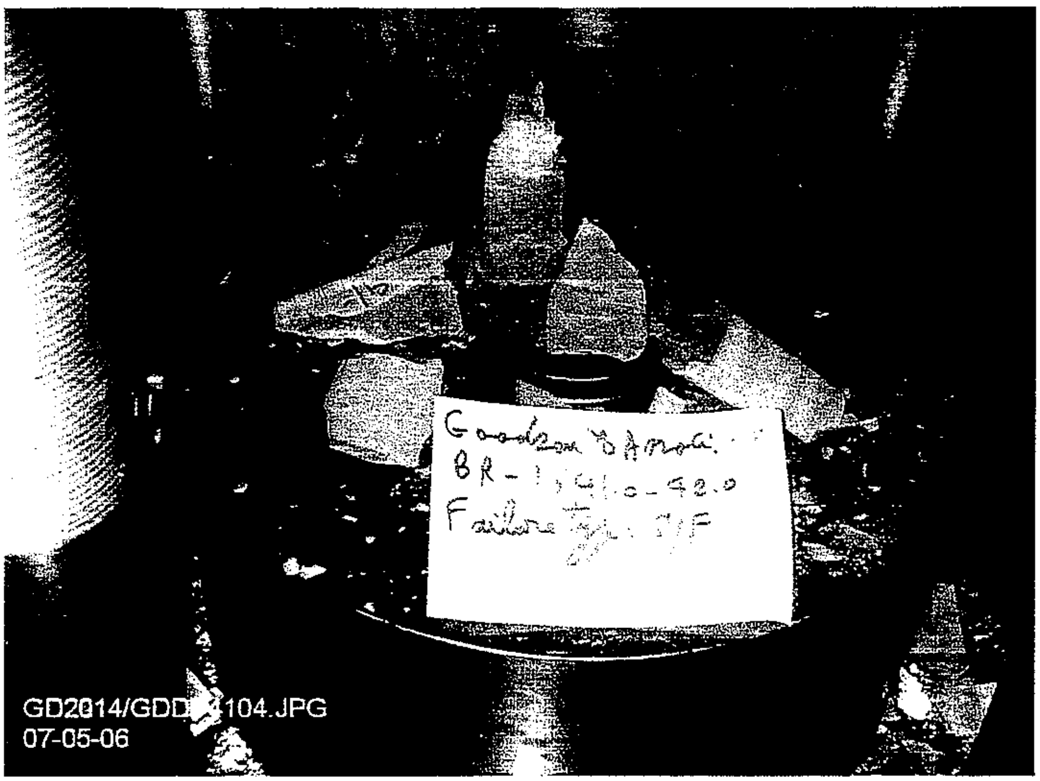
Notes and Comments: * Indicates L/D < 2.0. Correction Factor for short sample was applied.
C=Ca/[0.88+0.24b/h]
Ca = Failure Load / Surface Area
b = Sample Diameter
h = Sample Length

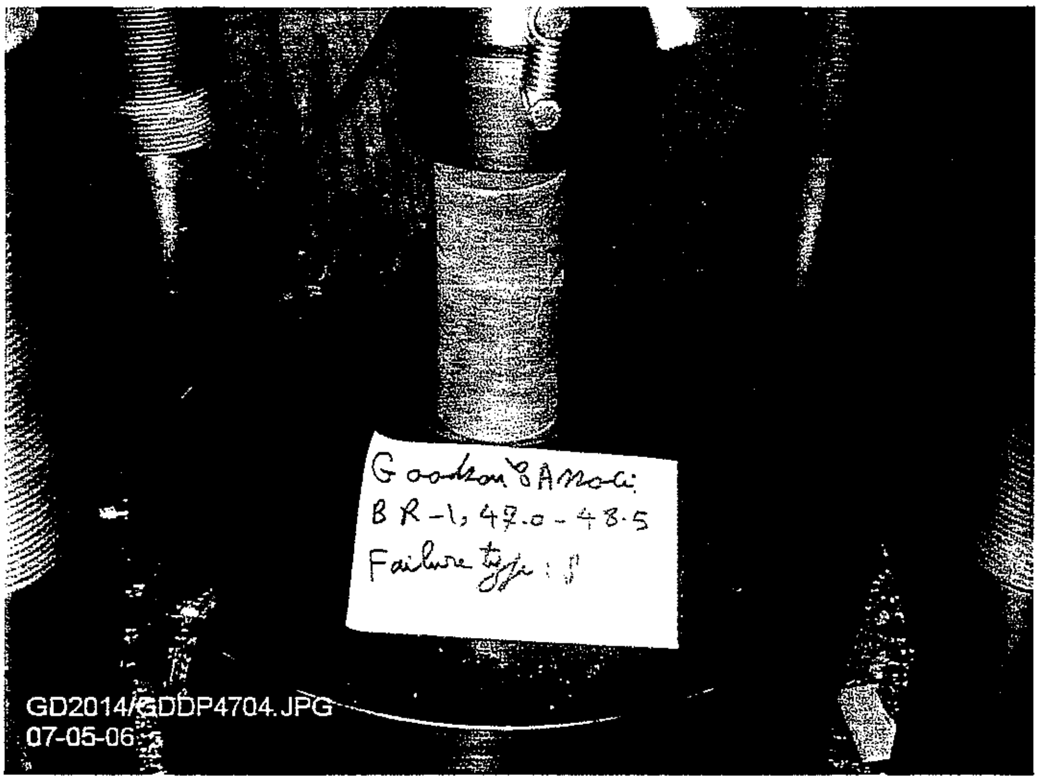
** Failure Type:
S: Shear Failure, M: Matrix Failure, F/V Fracture, Bedding/Void Collapse, C: Combination

*** Failure first occurred near the middle of sample.
+ Failure occured in soft portion of sample near bottom.

Data Entered By: HN Date: 09/28/2006
Data Checked By: (initials) Date: 09/28/06
Filename: GDUCSSR3

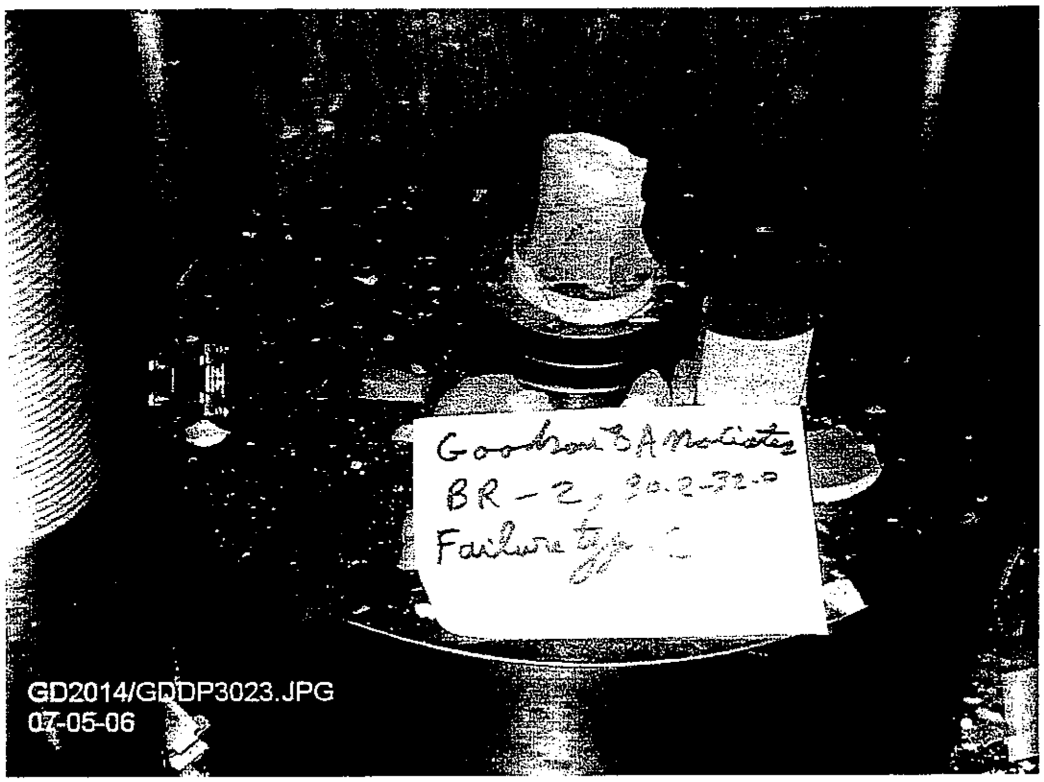
ADVANCED TERRA TESTING, Inc.

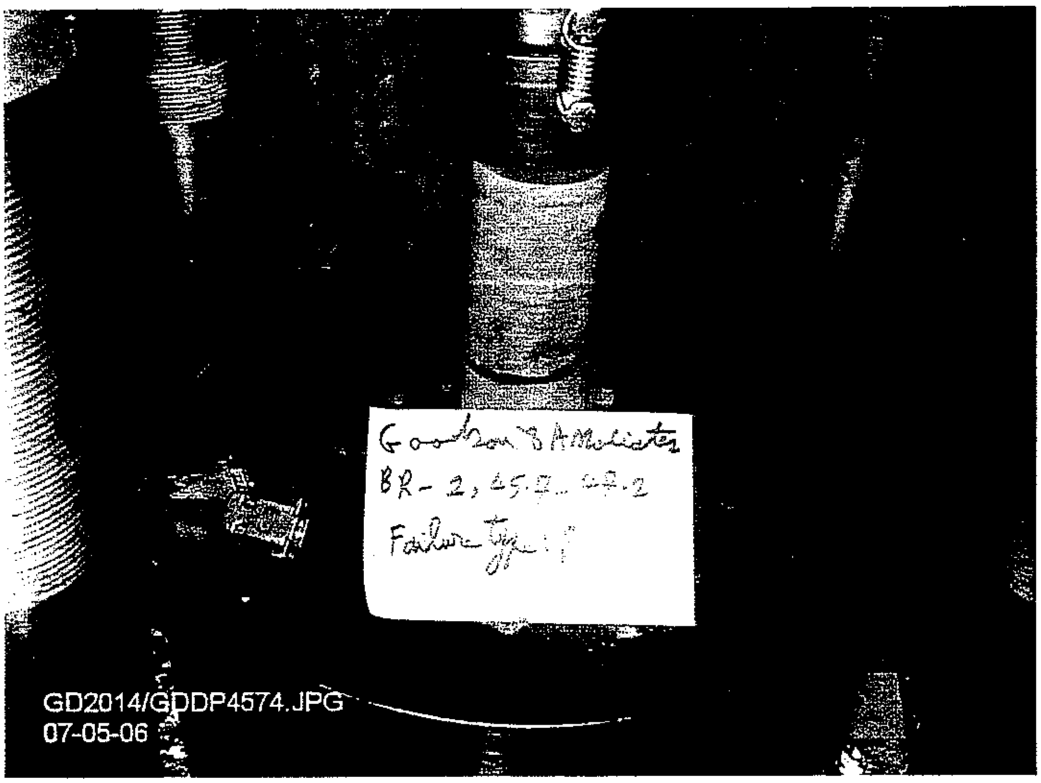

07-05-06


Goodson & Assoc:
BR-1, 47.0 - 48.5
Failure type:
GD2014/GDDP4704.JPG
07-05-06

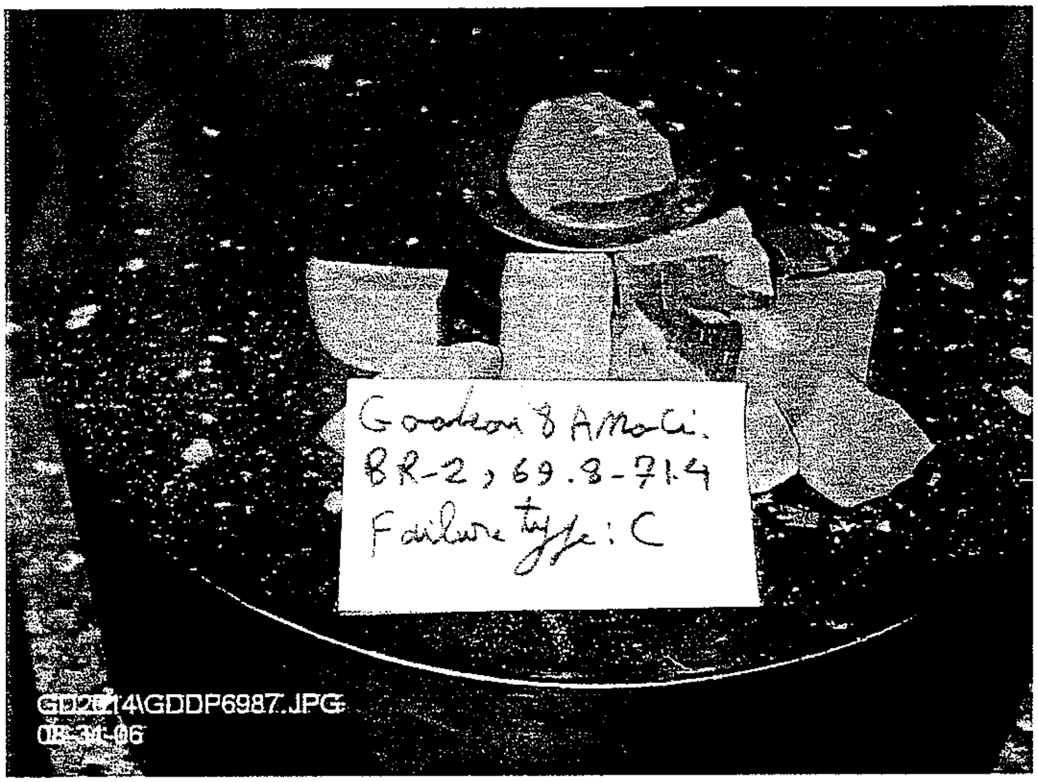

BR-1, 65.5-66.4
P6556.JPG

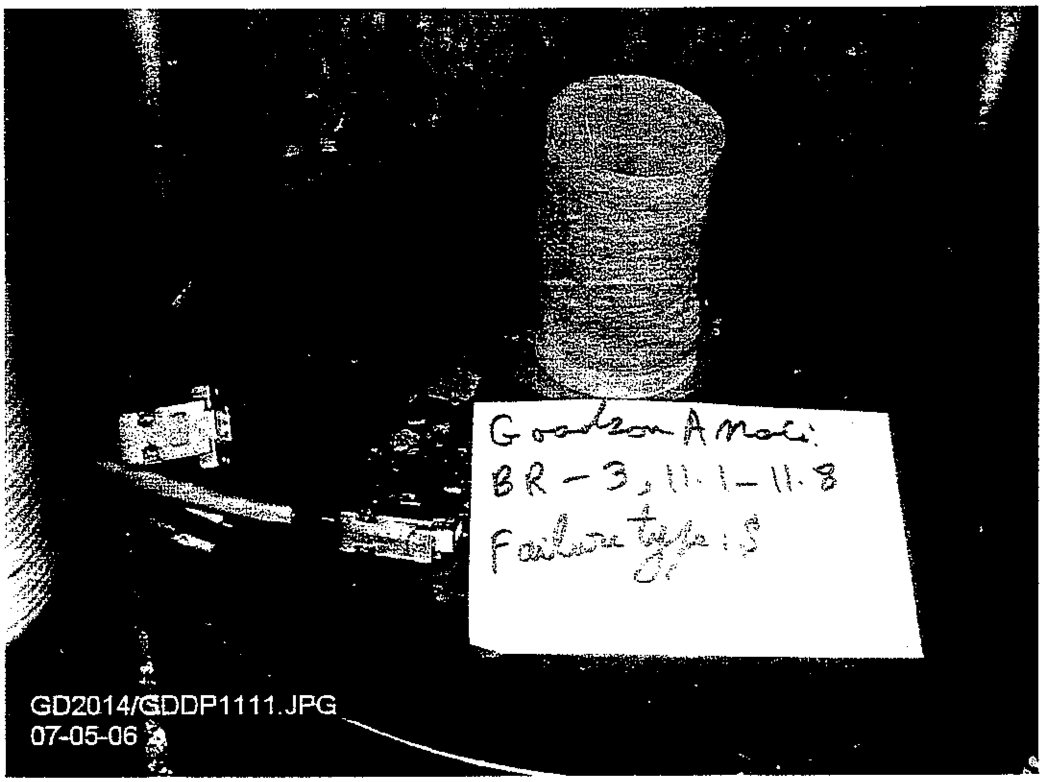

Goodson & Associates
BR-1, 100.0-101.9
Failure type: F
GD201AGDDP1000.JPG
08-29-06

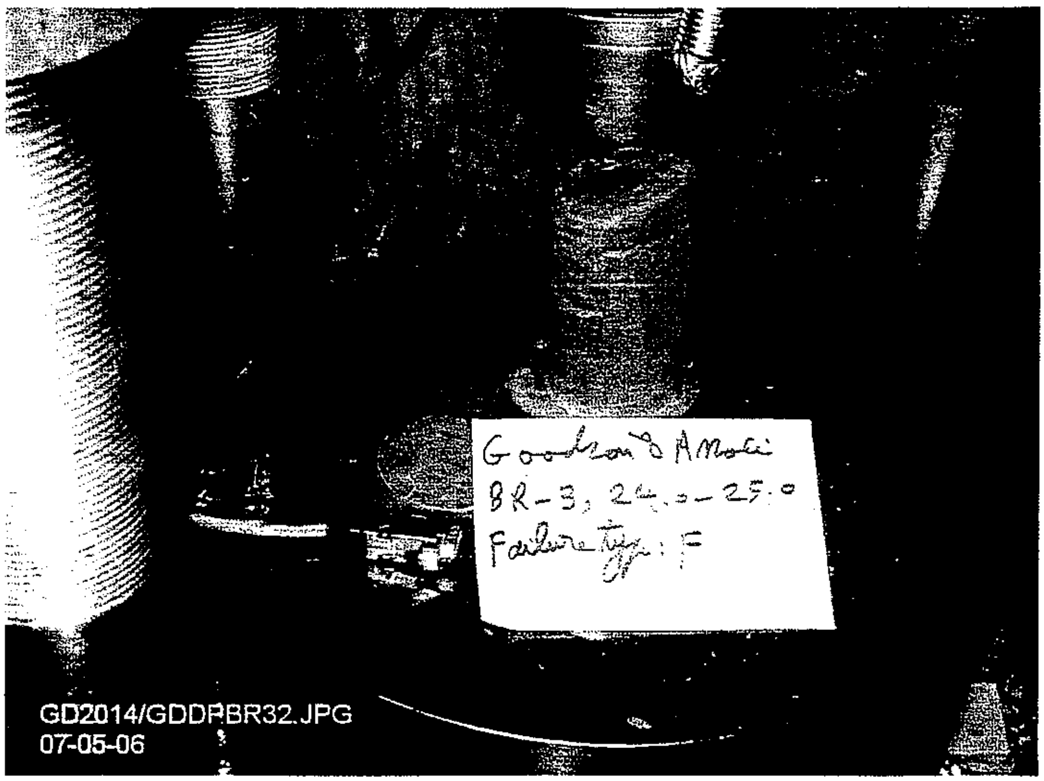

Goodson & Associates
BR-2, 30.2-32.0
Failure type C
GD2014/GDDP3023.JPG
07-05-06


Goodson & Associates
BR-2, 45.7-47.2
Failure Type
GD2014/GDDP4574.JPG
07-05-06


BR-2, 55.6-57.3
Failure type: S
08-29-06


Goodson & Amoli
BR-2, 69.8-71.4
Failure type: C
GDDP6987.JPG


Goodson A Moli
BR - 3, 11.1 - 11.8
Failure type: S
GD2014/GDDP1111.JPG
07-05-06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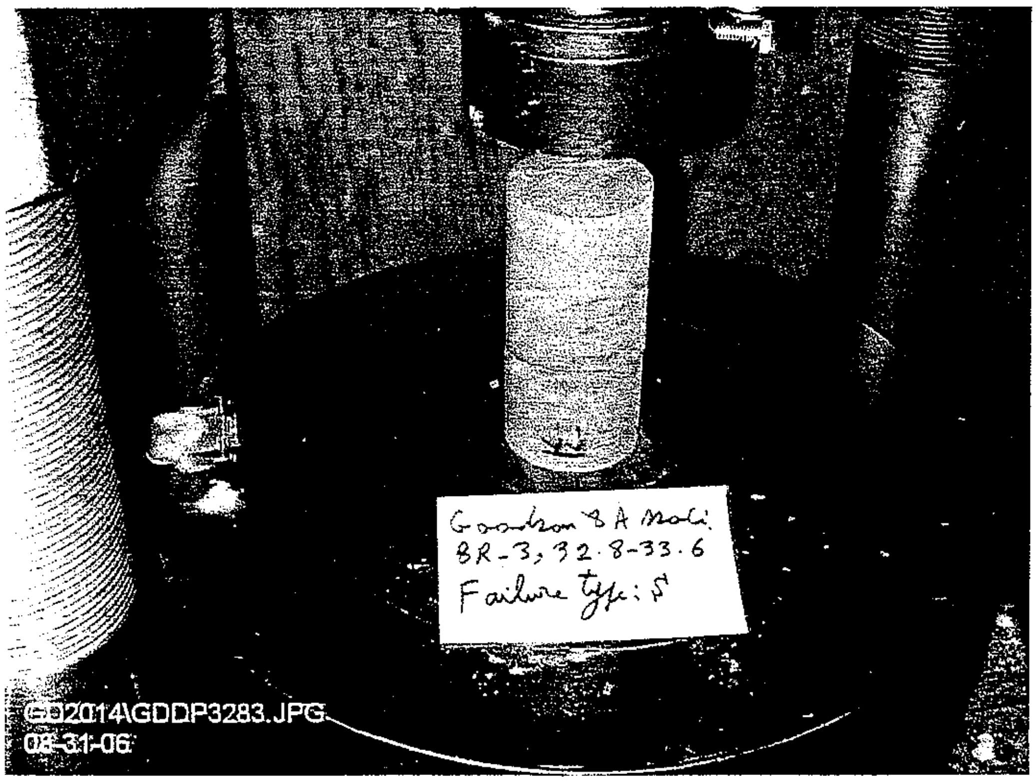
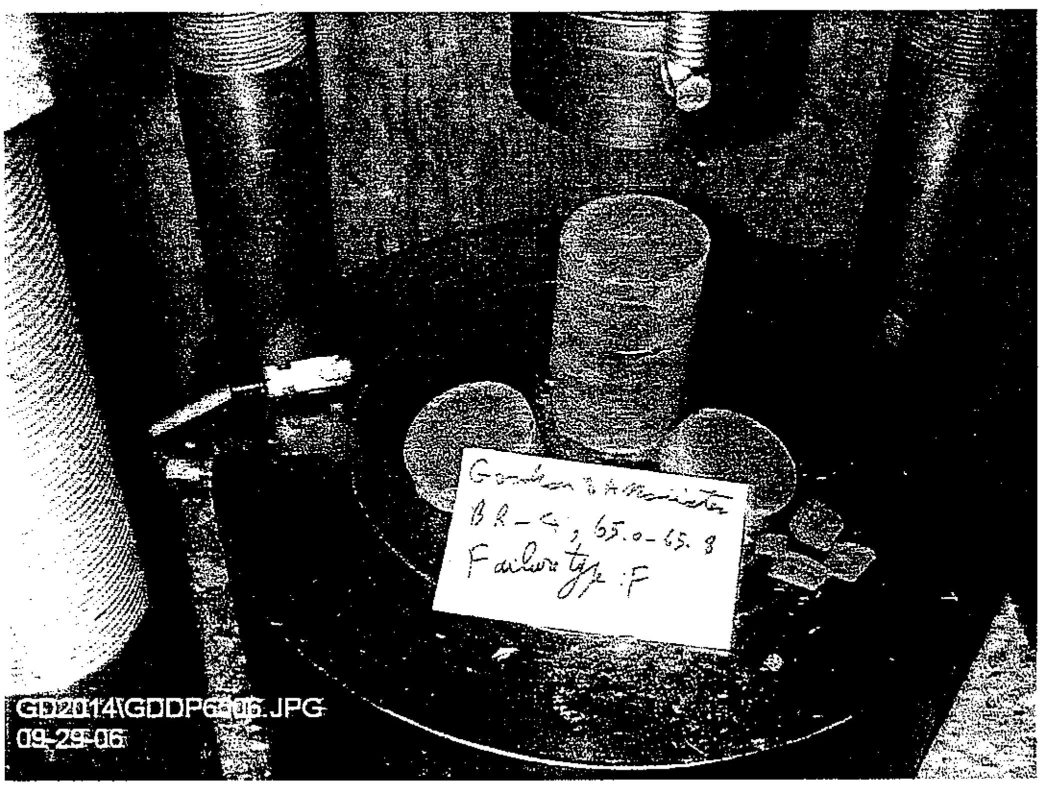
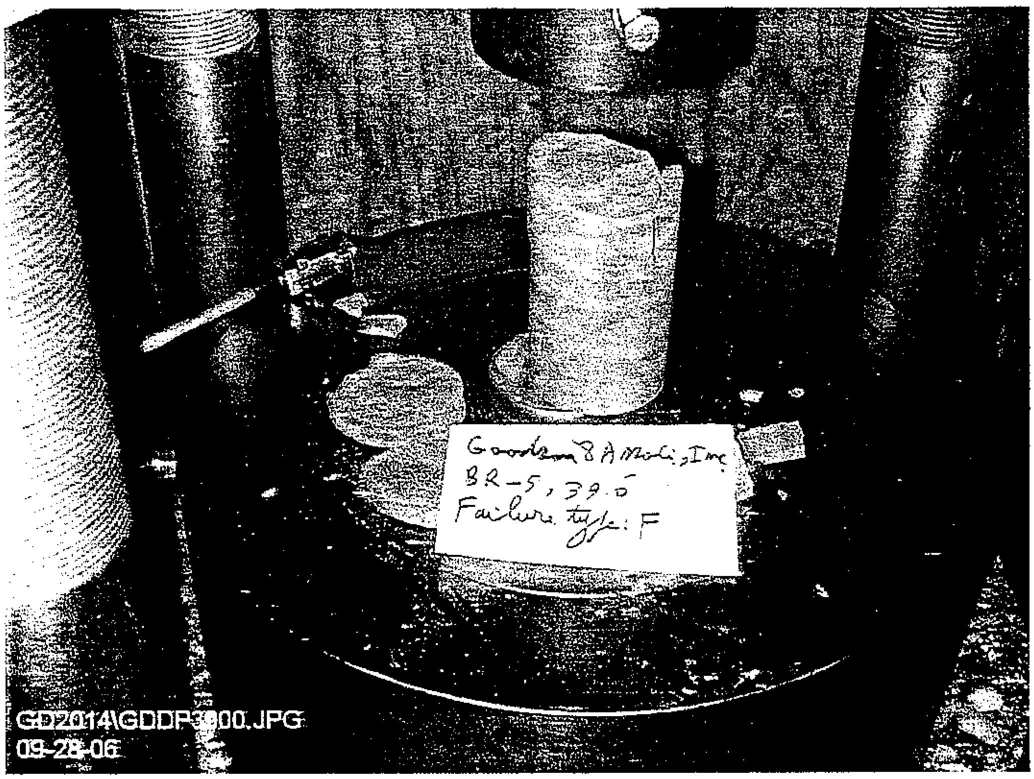
BR-3, 24.0-25.0
Failure Type: F
GD2014/GDDPBR32.JPG
07-05-06

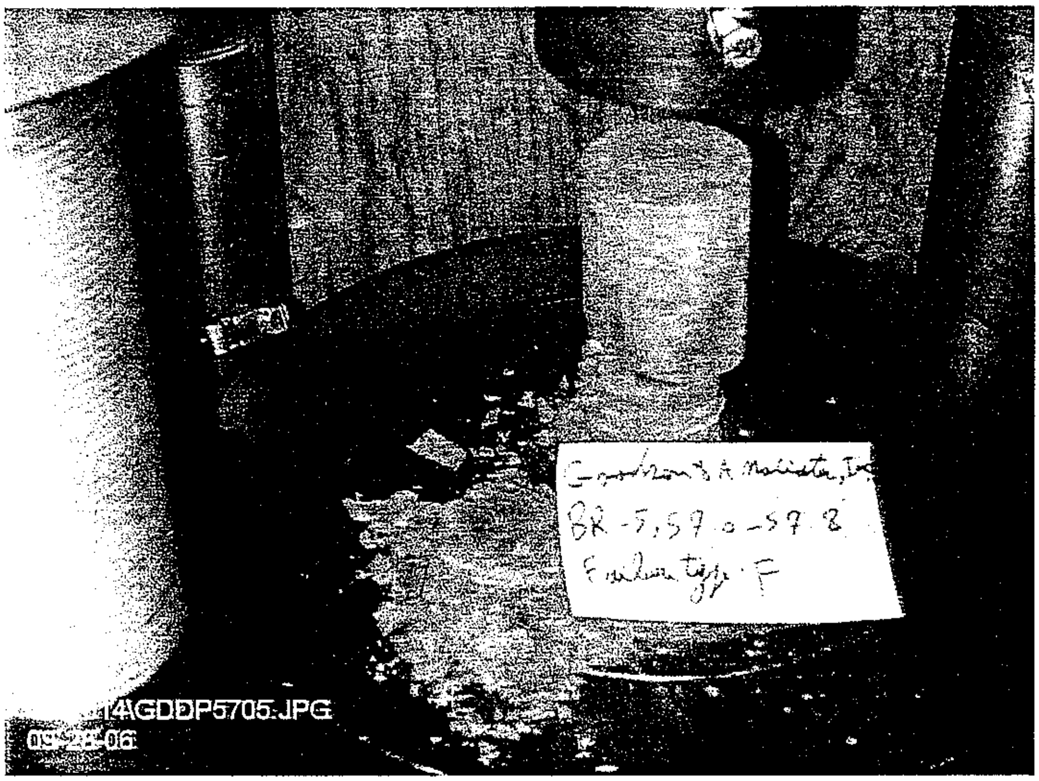

Goodson 8A Moli
BR-3, 32.8-33.6
Failure type: 5
GD2014\GDDP3283.JPG
08-31-06

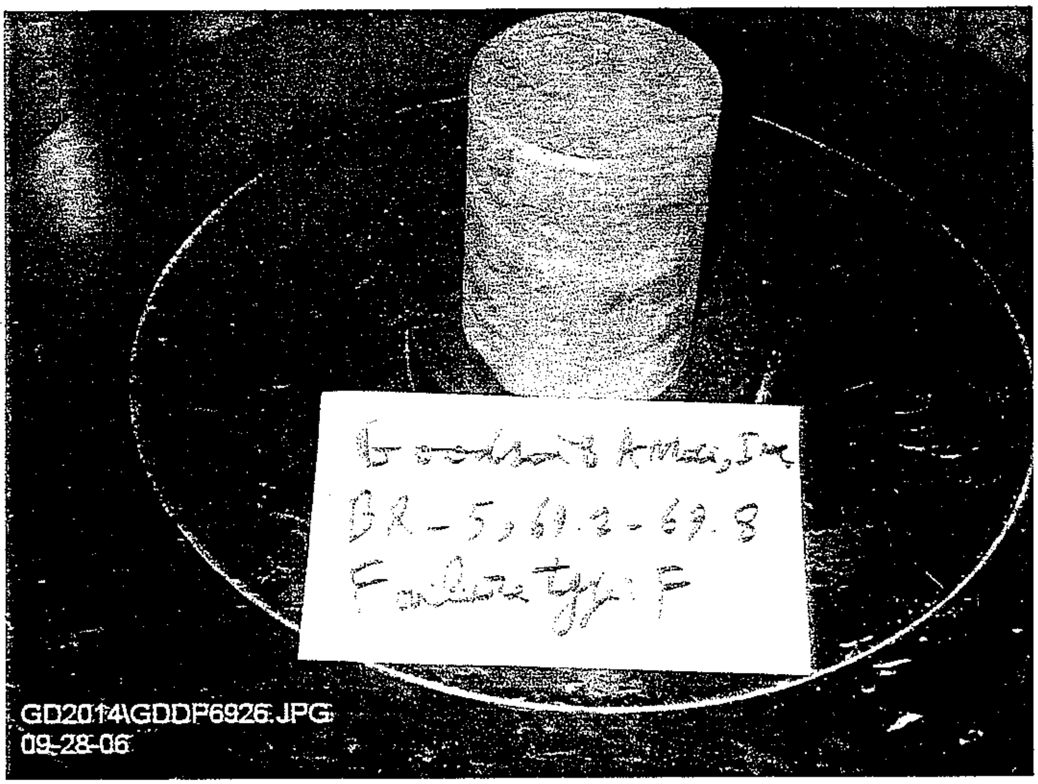

BR-9, 36.3-37.2
Failure type: F

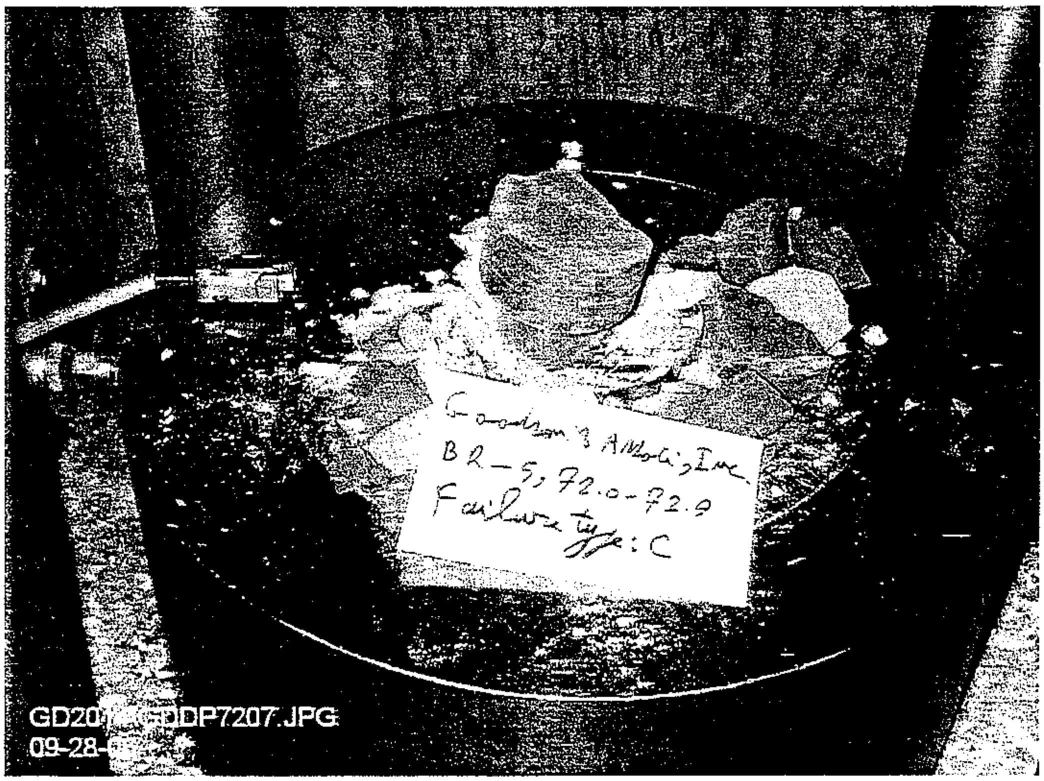
Failure type: F
GDDP4684.JPG
09-28-06


Gordon & Associates
BR-4, 65.0-65.8
Failure type: F
09-29-06


BR-5, 39.5'
Failure type: F
09-28-06


BR-5, 57.0-57.8'
Failure type: F
14\GDDP5705.JPG

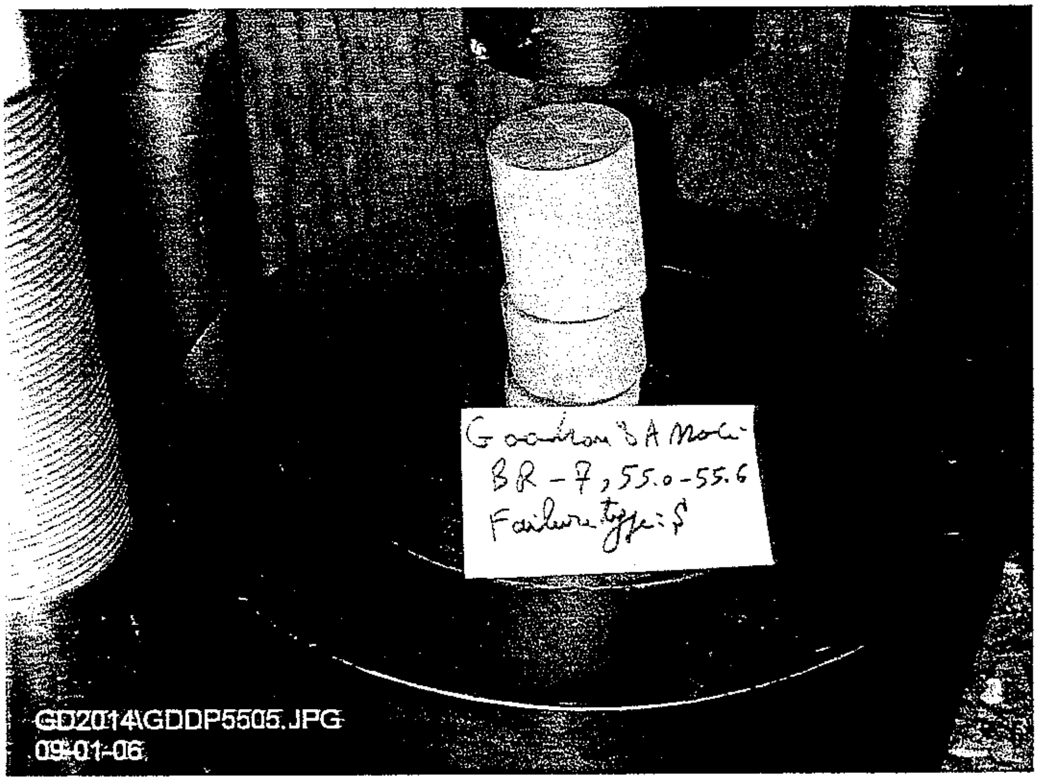

Failure Type: F
GD2014\GDDP6926.JPG
09-28-06


Goodson & Assoc., Inc.
BR-5, 72.0-72.9
Failure type: C
DDP7207.JPG
09-28-


Goodson 8 A Moli
BR-6, 28.5-30.5
Failure type: 5
GD2014\GDDP2853.JPG
31-06

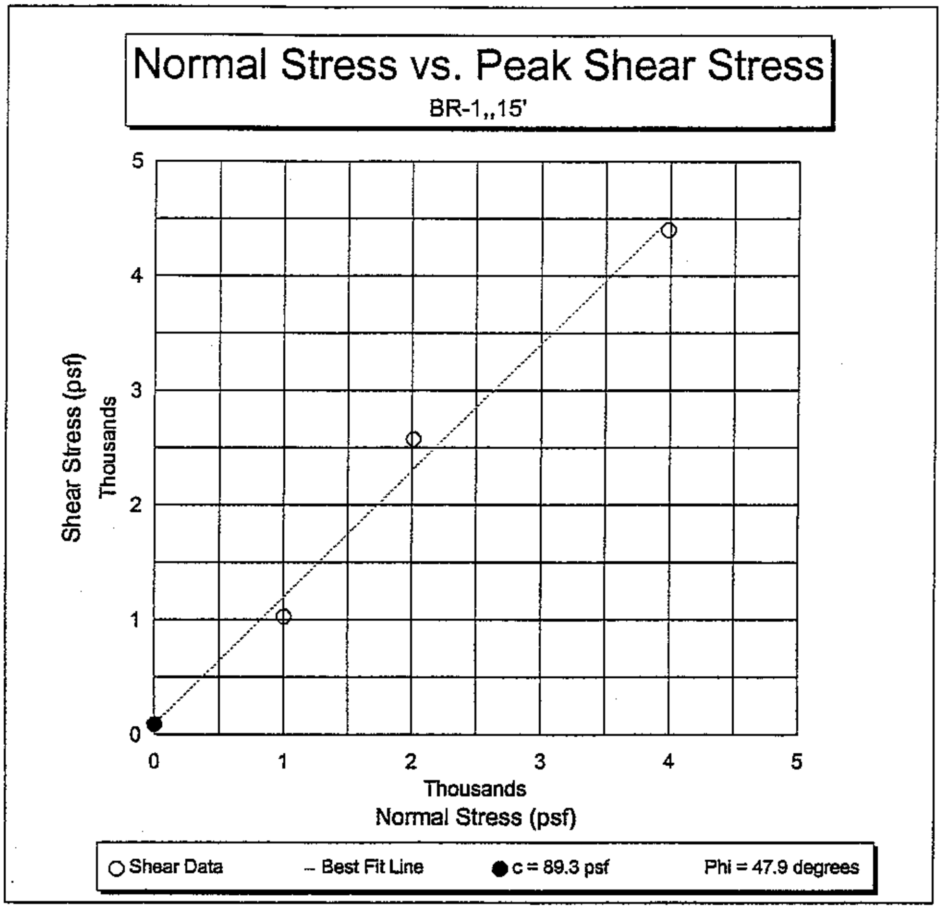

Goodson & Amoli
BR-6, 40.8-41.8
Failure type: S/F
GD2014\GDDP4084.JPG
08-31-06

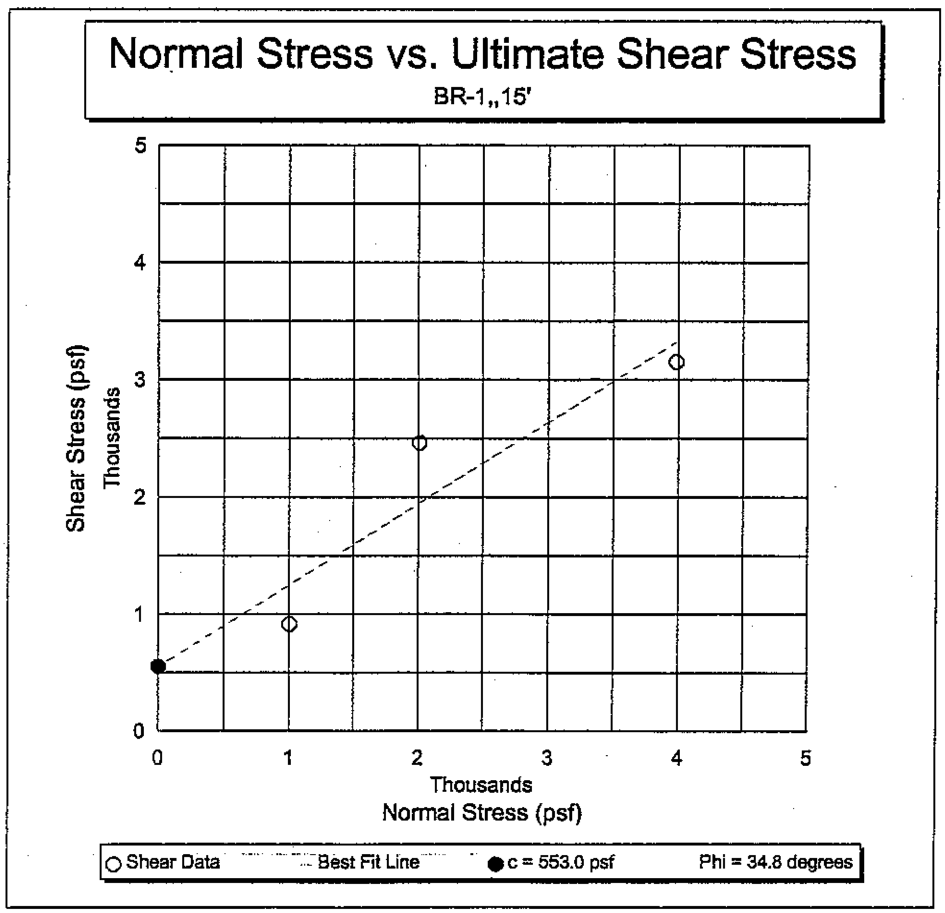

Goodson & Amoli
BR-7, 45.0-45.5
Failure type: F/S
GD2014\GDDP4504.JPG
08-29-06


Goodman & A Mok
BR-7, 55.0-55.6
Failure type: S
GD2014\GDDP5505.JPG
09-01-06


Goodison 8A Noli
BR-8, 48.8-50.0
Failure Type: F
GD2014\GDDP4885.JPG
09-01-06


BR-8, 61.0-62.0
Failure type: F
GD2014\GDDP6106.JPG
09-01-06



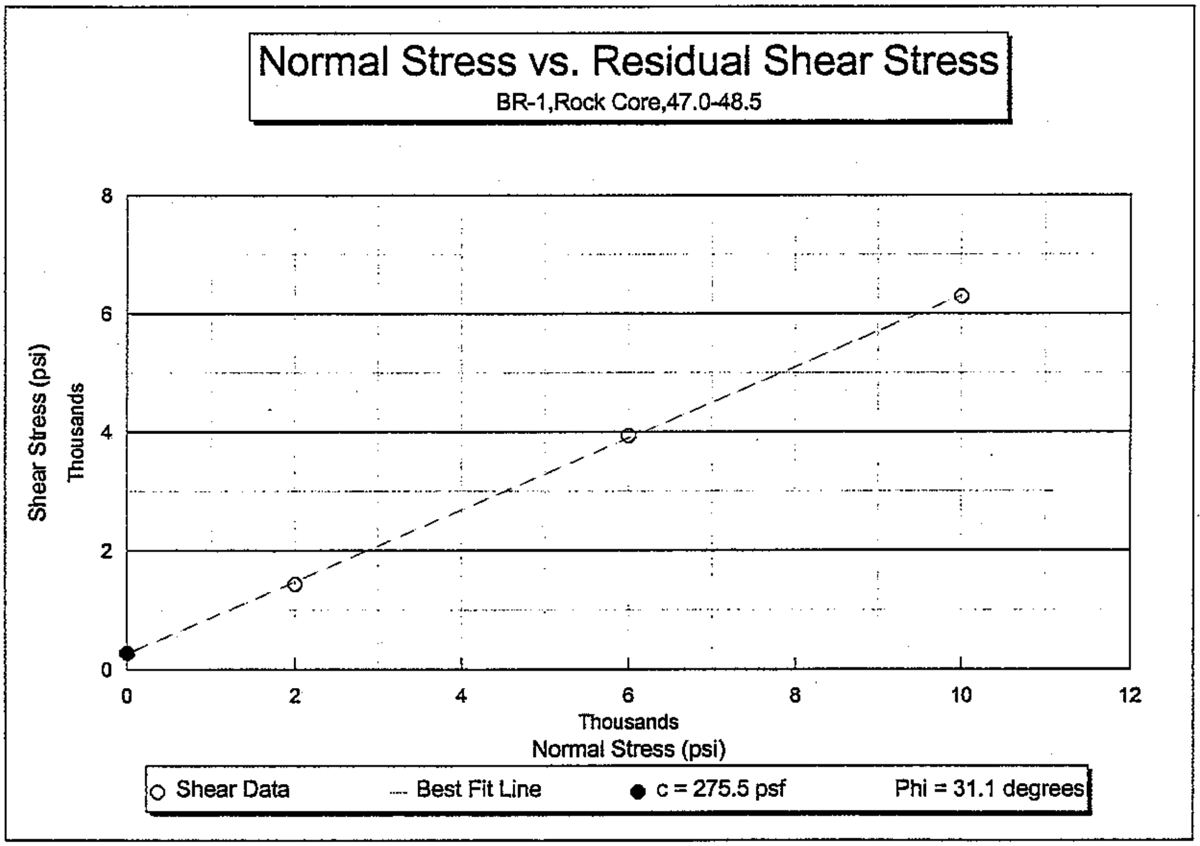


ADVANCED TERRA TESTING, INC.

JOINT DIRECT SHEAR
ASTM D 3080 Modified

CLIENT	Goodson & Associates	JOB NO.	2014-104
BORING NO.	BR-1	DATE SAMPLED	
DEPTH	47.0-48.5	DATE TESTED	06-21-06 DPM
SAMPLE NO.	Rock Core	LOCATION	4th Street Bridge, SH 96
ROCK TYPE	Sandy Siltstone	JOINT TYPE	Saw Cut

NORMAL STRESS (psf)	RESIDUAL SHEAR STRESS (psf)
10000	6292
6000	3940
2000	1430



Notes and Comments:

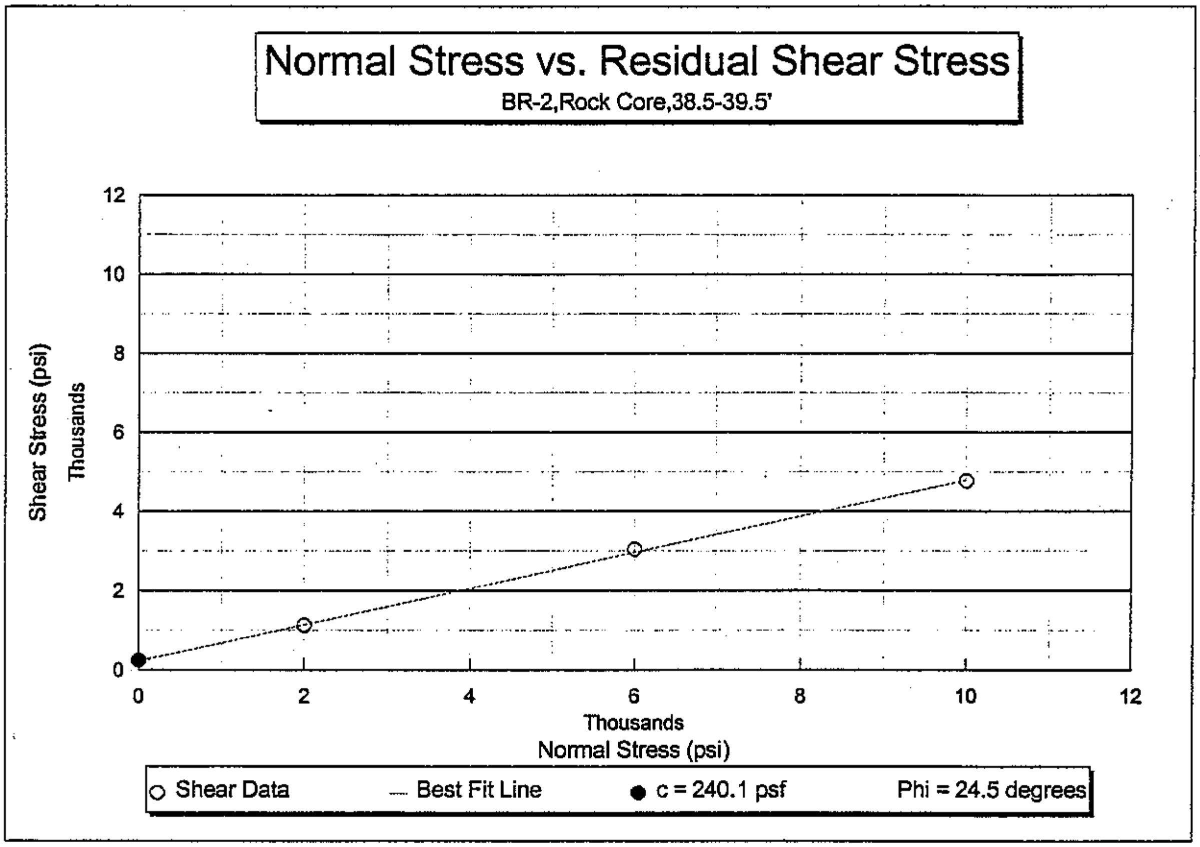
Data entered by: SR Date: 06/26/2006
Data checked by: DPM Date: 6/27/06
File Name: GDDSBR1

ADVANCED TERRA TESTING, inc.

JOINT DIRECT SHEAR
ASTM D 3080/MODIFIED

CLIENT	Godson & Associates	JOB NO.	2014-104
BORING NO.	BR-2	DATE SAMPLED	
DEPTH	38.5-39.5'	DATE TESTED	06-23,26&27-06 DPM/KB
SAMPLE NO.	Rock Core	LOCATION	4th Street Bridge, SH 96
ROCK TYPE		JOINT TYPE	Saw Cut

NORMAL STRESS (psf)	RESIDUAL SHEAR STRESS (psf)
10000	4767
6000	3039
2000	1120



Notes and Comments:

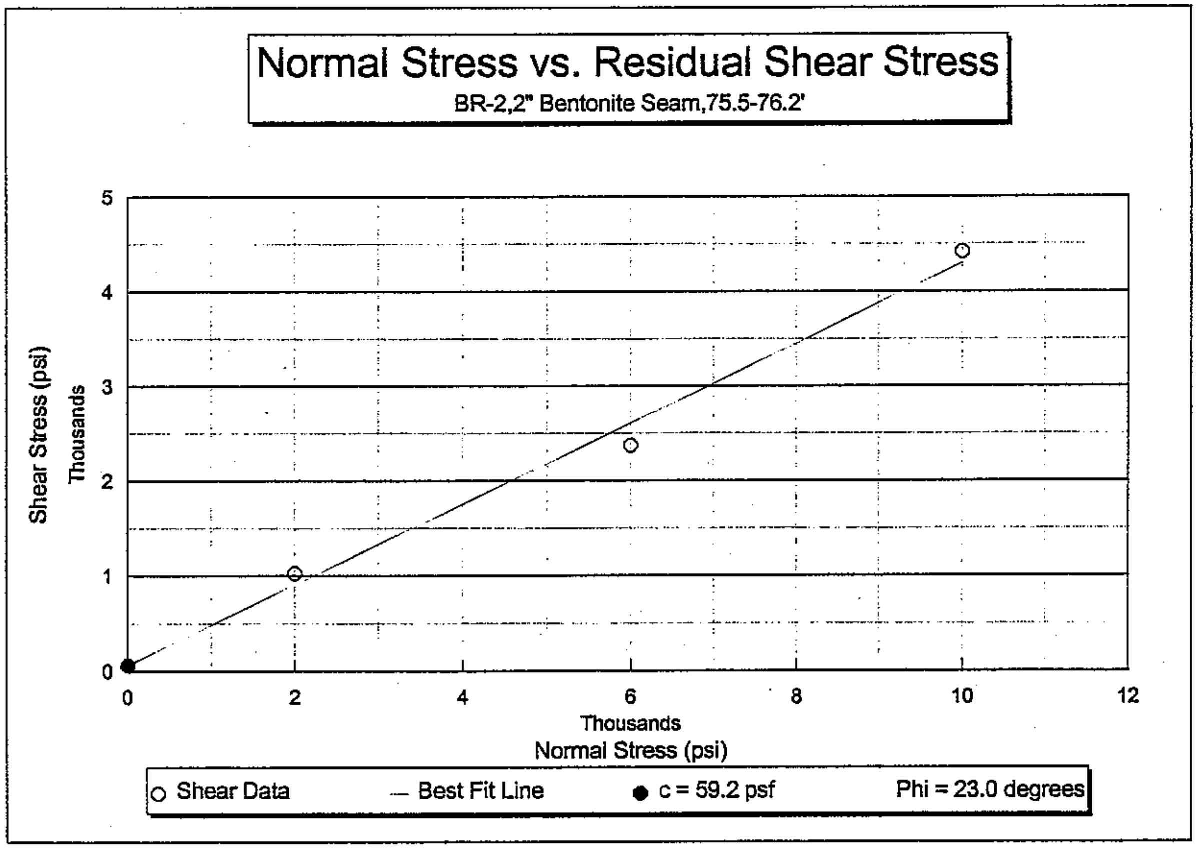
Data entered by: SR Date: 06/27/2006
Data checked by: RS Date: 7/17/06
File Name: GDDSBR2

ADVANCED TERRA TESTING, inc.

JOINT DIRECT SHEAR
Modified ASTM D 3080

CLIENT	Goodson & Associates	JOB NO.	2014-104
BORING NO.	BR-2	DATE SAMPLED	
DEPTH	75.5-76.2'	DATE TESTED	07-05-06 DPM
SAMPLE NO.	2" Bentonite Seam	LOCATION	4th Street Bridge SH 96
ROCK TYPE	Bentonite Seam	JOINT TYPE	Intact Rock

NORMAL STRESS (psf)	RESIDUAL SHEAR STRESS (psf)
10000	4418
6000	2369
2000	1025



Notes and Comments:

Data entered by: SR Date: 07/06/2006
Data checked by: DZ Date: 7/10/06
File Name: GDDSBR27

ADVANCED TERRA TESTING, inc.

SPLITTING TENSILE STRENGTH
By Method of Brazilian Disk
ASTM D 3967

CLIENT: Goodson & Associates, Inc. JOB NO.: 2014-104

LOCATION: DATE TESTED: 6/30/06 HN

PROJECT: 4th Street Bridge

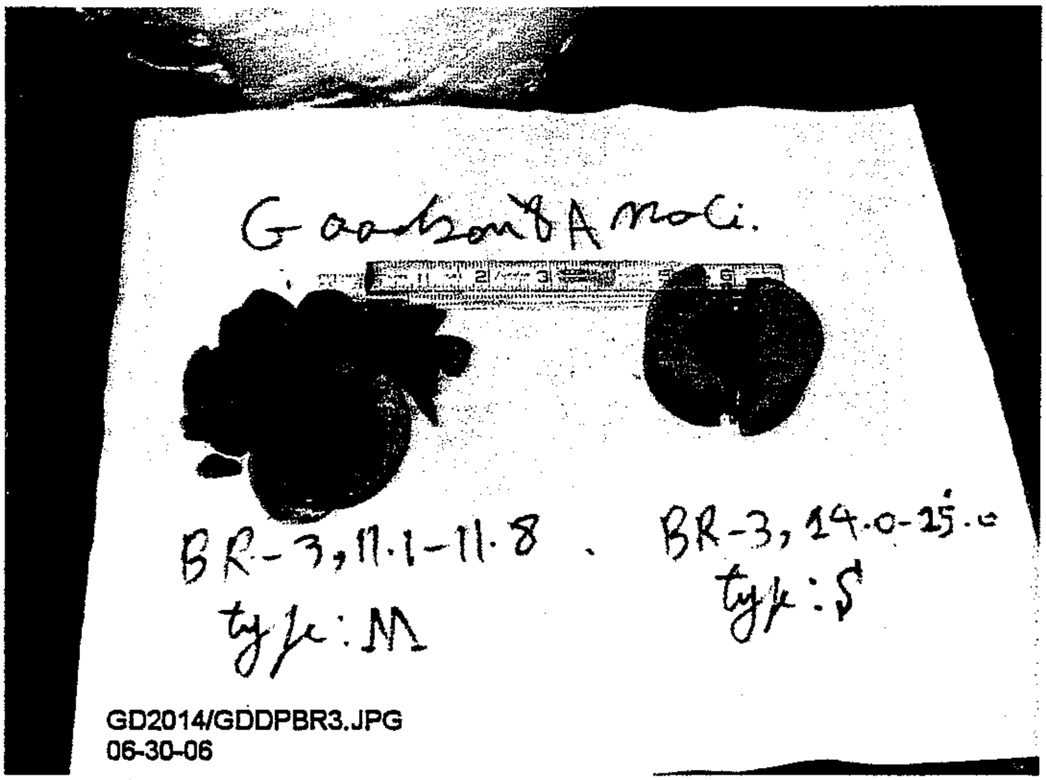
Specimen ID Boring, Depth(ft.)	Diameter (in.)	Length (in.)	Mass (gms)	Wet Density (pcf)	Failure Load (lb)	Failure Type *	Splitting Tensile Strength (psi)
BR-3, 11.1-11.8	2.406	1.212	202.20	139.8	1,115	M	240
BR-3, 24.0-25.0	2.408	1.190	211.80	148.9	3,090	S	690

Notes and Comments: Splitting Tensile Strength=2P/piLD.
P=Failure Load
pi = 3.1415926....
D = Sample Diameter
L = Sample Length

* Failure Type: S: Single Failure Plane, M: Multiple Failure Planes

Data Entered By: HN Date: 06/30/2006
Data Checked By: ω Date: 07/05/06
Filename: GDBRAZCP

ADVANCED TERRA TESTING,



GD2014/GDDPBR3.JPG
06-30-06

12200 West 50th Place
Wheat Ridge, CO 80033
(303) 233-2244

GOODSON ASSOCIATES
SUMMARY OF LABORATORY TEST RESULTS

Project Name: 4th Street Bridge
Project Number: 65569.01
Date: 10.2006

Boring Number	Depth (feet)	Nat. Moist. (%)	Nat. Dry Density (PCF)	Gravel (%)	Sand (%)	Fines (%)	Atterberg Limits LL	Atterberg Limits PI		Swell Consol. @ 500 PSF (%)	AASHTO Group Index	Unified Class.	SOIL DESCRIPTION
BR-1	0-5			55.7	27.9	16.4	NV	NP			A-1-b	GM	Silty Gravel with Sand
BR-1	10.0	13.9	102.0	0.0	22.9	77.1	52	25		0.2	A-7-6(20)	CH	Sandy Claystone
BR-1	15.0			0.0	0.8	99.2	43	20			A-7-6(22)	CL	Claystone
BR-1	42-42.2			0.0	16.9	83.1	NV	NP			A-4(0)	ML	Shale Bedrock
BR-1	47-48.5			0.0	42.4	57.6					A-4(0)	ML	Shale Bedrock
BR-1	65.5-66.4	1.6	145.9										Shale Bedrock
BR-2	5.0	2.0	112.3	8.7	56.6	34.7	NV	NP		- 2.7	A-2-4(0)	SM	Silty Sand Fill
BR-2	38.5-39.5			0.0	73.1	26.9	27	12					Shale Bedrock
BR-2	55.6-57.3	1.9	139.4										Shale Bedrock
BR-2	75.5-76.2			0.0	18.1	81.9	51	36			A-7-6(29)	CH	Shale Bedrock
BR-3	5.0	17.0		23.6	40.1	36.3	32	18			A-6(2)	SC	Clayey Sand with Gravel
BR-3	32.8-33.6	1.0	143.4										Shale Bedrock
BR-4	0-5			48.2	34.6	17.2	28	5			A-1-b	GM	Silty Gravel with Sand
BR-4	10.0	4.8		68.9	29.9	1.2	NV	NP			A-1-a	GP	Poorly Graded Gravel with Sand
BR-4	36.3-37.2	2.9	143.4										Shale Bedrock
BR-4	46.8-48.2	2.4	146.3	0.0	63.1	36.9	28	10					Shale Bedrock
BR-4	55.5-57.5	6.9	141.1										Shale Bedrock
BR-5	1-5			33.5	41.6	24.9	35	4			A-1-b	SM	Silty Sand with Gravel
BR-5	10.0	4.0	136.0	61.2	37.9	0.9	NV	NP			A-1-a	GP	Poorly Graded Gravel with Sand
BR-5	38.0-39.5			0.0	82.7	17.3	27	10					Shale Bedrock
BR-5	39	2.2	145.8										Shale Bedrock
BR-5	69.2-69.6	2.4	141.5										Shale Bedrock
BR-6	5.0	12.7	101.8	45.2	19.3	35.5	43	27			A-7-6(4)	GC	Clayey Gravel with Sand
BR-6	28.5-30.5			0.0	36.2	63.8	29	12					Shale Bedrock
BR-6	40.8-41.8	1.0	147.9										Shale Bedrock
BR-7	10.0	14.5		16.7	49.4	33.9	NV	NP			A-2-4(0)	SM	Silty Sand with Gravel
BR-7	15.0	14.6	107.2	0.0	33.9	66.1	NV	NP			A-4(0)	ML	Sandy Silt
BR-7	30.0	12.7		11.7	83.6	4.7	NV	NP			A-1-b	SP	Poorly Graded Sand
BR-7	45.0-45.5	1.1	143.9										Shale Bedrock
BR-7	55.0-55.6			0.0	63.5	36.5	31	14					Shale Bedrock
BR-8	1.5	12.1	116.2	8.9	18.9	72.2	40	19		0.8	A-6(13)	CL	Lean Clay with Sand
BR-8	10.0	14.3	113.4	6.8	15.8	77.4	45	20		- 0.4	A-7-6(16)	CL	Lean Clay with Sand
BR-8	30.0	12.4	98.0	0.0	93.8	6.2	NV	NP			A-3	SP-SM	Poorly Graded Sand with Silt

Note: Percent fines test results for BR-2 at 38.5 to 39.5 feet, BR-4 at 46.8 to 48.2 feet, BR-5 at 38.0 to 39.5 feet, and BR-7 at 55.0 to 55.6 feet appear to be low likely due to difficulty breaking down shale rock fragments.

Table 1

12200 West 50th Place
Wheat Ridge, CO 80033
(303) 233-2244

GOODSON ASSOCIATES
SUMMARY OF LABORATORY TEST RESULTS

Project Name: 4th Street Bridge
Project Number: 65569.01
Date: 10.2006

Boring Number	Depth (feet)	Nat. Moist. (%)	Nat. Dry Density (PCF)	Gravel (%)	Sand (%)	Fines (%)	Atterberg Limits LL	Atterberg Limits PI	R Value	Swell Consol. @ 200 PSF (%)	AASHTO Group Index	Unified Class.	SOIL DESCRIPTION
DC-1	0-5			43.5	40.3	16.2	NV	NP			A-1-b	GM	Silty Gravel with Sand
DC-1	15.0	17.4	108.3	0.0	1.3	98.7	48	23		4.0	A-7-6(26)	CL	Claystone
DC-1	20.0	17.8	112.9										Claystone
DF-1	1.5	17.8	88.5	0.0	3.2	96.8	41	20		- 0.5	A-7-6(21)	CL	Lean Clay
DF-1	5.0	15.0	107.4							0.9			Claystone
DF-2	5.0	13.0	110.8	0.4	36.2	63.4	28	12			A-6(5)	CL	Sandy Lean Clay
DF-2	15.0	13.8	93.8	0.5	63.2	36.3	NV	NP			A-4(0)	SM	Silty Sand
DF-2	20.0	10.3		46.6	44.7	8.7	NV	NP			A-1-b	GP-GM	Poorly Graded Gravel with Silt and Sand
DF-3	1.5	16.6	100.0	1.2	56.4	42.4	24	6			A-4(0)	SC-SM	Silty Clayey Sand
DF-3	5.0	6.0		38.8	53.3	7.9	NV	NP			A-1-b	SP-SM	Poorly Graded Sand with Silt and Gravel
DF-3	15.0	8.5		33.6	58.6	7.8	NV	NP			A-1-b	SP-SM	Poorly Graded Sand with Silt and Gravel
DF-4	5.0	15.4	105.2	0.8	36.2	63.0	34	15			A-6(7)	CL	Sandy Lean Clay
DF-4	15.0	14.3		25.4	64.1	10.5	NV	NP			A-1-b	SP-SM	Poorly Graded Sand with Silt and Gravel
DF-5	1.5	12.5	91.6	0.5	34.7	64.8	30	13			A-6(6)	CL	Sandy Lean Clay
DF-5	5.0	7.2	93.7	0.0	77.1	22.9	NV	NP			A-2-4(0)	SM	Silty Sand
DF-5	20.0	9.5		46.0	47.4	6.6	NV	NP			A-1-a	SP-SM	Poorly Graded Sand with Silt and Gravel

Table 1

12200 West 50th Place
Wheat Ridge, CO 80033
(303) 233-2244

GOODSON ASSOCIATES
SUMMARY OF LABORATORY TEST RESULTS

Project Name: 4th Street Bridge
Project Number: 65569.01
Date: 10.2006

Boring Number	Depth (feet)	Nat. Moist. (%)	Nat. Dry Density (PCF)	Gravel (%)	Sand (%)	Fines (%)	Atterberg Limits LL	Atterberg Limits PI	R Value	Swell Consol. @ 200 PSF (%)	AASHTO Group Index	Unified Class.	SOIL DESCRIPTION
P-1	1-5			10.4	46.0	43.6	24	5	67		A-4(0)	SC-SM	Silty Clayey Sand
P-2	0-5			6.5	39.4	54.1	37	16	48		A-6(6)	CL	Sandy Lean Clay
P-2	5.0	18.3	110.4	0.0	3.6	96.4	48	21		3.0	A-7-6(24)	CL	Claystone
P-3	1-5			1.5	33.2	65.3	33	15	26		A-6(8)	CL	Sandy Lean Clay
P-4	1-5			35.7	51.4	12.9	NV	NP			A-1-b	SM	Silty Sand with Gravel
P-4	5.0	19.5	100.4	0.0	39.8	60.2	28	7			A-4(2)	CL-ML	Sandy Silty Clay
P-5	1-5			3.6	58.7	37.7	24	4	73		A-4(0)	SC-SM	Silty Clayey Sand
P-6	1-5			32.4	48.1	19.5	NV	NP	66		A-1-b	SM	Silty Sand with Gravel

Table 1

12200 West 50th Place
Wheat Ridge, CO 80033
(303) 233-2244

GOODSON ASSOCIATES
SUMMARY OF LABORATORY TEST RESULTS

Project Name: 4th Street Bridge
Project Number: 65569.01
Date: 10.2006

Boring Number	Depth (feet)	Nat. Moist. (%)	Nat. Dry Density (PCF)	Gravel (%)	Sand (%)	Fines (%)	Atterberg Limits LL	Atterberg Limits PI	Unconfined Comp. (PSF)	Swell Consol. @ 200 PSF (%)	AASHTO Group Index	Unified Class.	SOIL DESCRIPTION
ET-2	0-7			19.9	53.9	26.2	NV	NP			A-2-4(0)	SM	Silty Sand with Gravel
ET-2	9-11			16.0	36.8	47.2	32	12			A-6(3)	SC	Clayey Sand with Gravel

Table 1

12200 West 50th Place
Wheat Ridge, CO 80033
(303)233-2244

Goodson and Associates
Summary of Laboratory Test Results

Project Name: 4th Street Bridge
Project Number: 65569.01
Date: 10.2006

Boring Number	Depth (Feet)	Water Soluble Sulfates (%)	pH	Chlorides (%)	Electrical Resistivity (OHM-CM)		
BR-1	0-5	0.0300	7.5	0.00002	1,100		
BR-1	65.5-66.4	0.2075	6.9	0.10500	300		
BR-2	55.6-57.3	0.0450	7.4	0.05250	200		
BR-3	32.8-33.6	0.2275	7.4	0.20500	200		
BR-4	0-5	0.2125	8.2	0.01813	300		
BR-4	46.8-48.2	0.1625	7.7	0.06250	200		
BR-5	1-5	0.0325	7.1	0.00175	1,100		
BR-5	38-39.5	0.2250	4.6	0.16250	200		
BR-6	28.5-30.5	0.2325	8.1	0.00004	200		
BR-7	10	0.1950	7.5	0.00079	200		
BR-7	45.0-45.5	0.1200	7.7	0.00019	300		
DC-1	0-5	0.0600	7.8	0.00120	700		
DC-1	20	0.1925	7.6	0.00006	400		

Table 2

APPENDIX C

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Goodson & Associates, Inc.
12200 W. 50th Place, Unit A
Wheat Ridge, CO 80033

Flexible Structural Design Module

SH 96A, 4th Street Bridge, R-Value=60

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	3,973,456
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	95 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	18,259 psi
Stage Construction	1
Calculated Design Structural Number	3.20 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	HMA	0.44	1	7.28	24	3.20
Total	-	-	-	7.28	-	3.20

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Goodson & Associates, Inc.
12200 W. 50th Place, Unit A
Wheat Ridge, CO 80033

Flexible Structural Design Module

SH 96A, 4th Bridge Street, R-Value=48

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	3,973,456
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	95 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	12,335 psi
Stage Construction	1
Calculated Design Structural Number	3.68 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	HMA	0.44	1	8.36	24	3.68
Total	-	-	-	8.36	-	3.68

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare Computer Software Product

Goodson & Associates, Inc.
12200 W. 50th Place, Unit A
Wheat Ridge, CO 80033

Flexible Structural Design Module

SH 96A, 4th Street Bridge, R-Value=26

Flexible Structural Design

18-kip ESALs Over Initial Performance Period	3,973,456
Initial Serviceability	4.5
Terminal Serviceability	2.5
Reliability Level	95 %
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus	6,010 psi
Stage Construction	1
Calculated Design Structural Number	4.70 in

Specified Layer Design

Layer	Material Description	Struct Coef. (Ai)	Drain Coef. (Mi)	Thickness (Di)(in)	Width (ft)	Calculated SN (in)
1	HMA	0.44	1	10.68	24	4.70
Total	-	-	-	10.68	-	4.70

APPENDIX D

Michael W. West and Associates, Inc.
Consulting Engineers and Geologists

Geological, Geotechnical, Environmental and Earthquake Engineering

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www.m-west-assoc.com

ENGINEERING GEOLOGIC INVESTIGATION AND SLOPE STABILITY EVALUATION, FINAL DESIGN PHASE, PROPOSED 4TH STREET (COLORADO STATE HIGHWAY 96A) BRIDGE OVER THE ARKANSAS RIVER PUEBLO, PUEBLO COUNTY, COLORADO CDOT PROJECT NO. BR0961-008/13141

Prepared for:

Goodson & Associates, Inc.
Attn: Mr. Dave Nasiatka, P.E.
12090 West 50th Place, Unit A
Wheat Ridge, CO 80033

January 25, 2007

Project No. 05675

ENGINEERING GEOLOGIC INVESTIGATION AND SLOPE STABILITY EVALUATION, FINAL DESIGN PHASE, PROPOSED 4TH STREET BRIDGE OVER THE ARKANSAS RIVER, PUEBLO, PUEBLO COUNTY, COLORADO

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ENGINEERING GEOLOGIC INVESTIGATION AND SLOPE STABILITY EVALUATION, FINAL DESIGN PHASE, PROPOSED 4TH STREET BRIDGE OVER THE ARKANSAS RIVER, PUEBLO, PUEBLO COUNTY, COLORADO

1. INTRODUCTION

The following report summarizes results of an engineering geologic investigation and slope stability evaluation of the southwest abutment for the proposed Colorado State Highway 96A (4th Street) Bridge in Pueblo, Pueblo County, Colorado. The new bridge will essentially parallel the existing bridge and will pass over, from northeast to southwest, a two-lane road (the loop ramp), the Union Pacific and Burlington Northern/Santa Fe Railroad Yard, a floodwall along the northeast side of the Arkansas River, and the Arkansas River. The riverbank at the proposed southwest abutment is approximately 70 feet high with the upper undisturbed portion of the existing riverbank slope at approximately 1.1h:1v. The proposed southwest abutment for the new 4th Street Bridge will be located approximately 28 feet northwest of the existing abutment.

This report presents results of the engineering geologic investigation including a geologic reconnaissance, estimation of the headcut erodibility index, and slope stability analyses to evaluate the stability of the riverbank slope at the proposed southwest abutment. In addition, this report provides recommendations for fill slopes, as well as other design and construction considerations based on extant geologic conditions.

1.1 OBJECTIVES

The objectives of our geological and slope stability studies were to: (1) assess geological and geotechnical conditions in the area of the proposed southwest abutment; (2) assess the probable impact of those conditions on project design;

(3) estimate the headcut erodibility index for riverbed materials; (4) assess the stability of the riverbank slope; and (5) develop design recommendations for the abutment location, abutment slope design, fill slopes, and related geological/geotechnical design issues.

1.2 SCOPE OF STUDIES

Our studies included: (1) a review of available geologic and geotechnical information for the site; (2) a geologic reconnaissance of the site including a detailed line survey of bedrock discontinuities and logging selected exploratory borings and an exploratory trench; (3) estimation of the headcut erodibility index for the bedrock materials in the river bottom area at Pier 2; (4) slope stability analyses for the proposed southwest abutment; (5) evaluation of the results of the field investigations, engineering analyses, and development of geologic and geotechnical design recommendations; and (6) preparation of a report signed by a Professional Engineer describing the data collected, methodologies employed, and conclusions, supported by appropriate illustrations.

A location map illustrating the existing features, the proposed location for the southwest abutment of the 4th Street Bridge, and the geology of the area is provided on Figure 1. Figures 2 and 3 illustrate the trench logs for Exploratory Trench ET-1. Results of the slope stability analyses are presented on Figures 4 through 13. A summary of laboratory test results is provided in Table 1. Rock structure information, developed from the detailed line survey, is summarized in Table 2, and a summary of the estimated Headcut Erodibility Index is provided in Table 3. Laboratory test results are provided in Attachment A.

2. SITE CONDITIONS AND PROPOSED CONSTRUCTION

2.1 SITE CONDITIONS

The 4th Street Bridge project is located in the northwest ¼ of Section 35, Township 20 South, Range 65 West of the 6th Prime Meridian. The new bridge will be located west of, and essentially parallel to the existing bridge. The southwest abutment of the new bridge is located approximately 28 feet northwest of the existing bridge abutment. We understand that the existing bridge will be removed.

The existing bridge was constructed in the 1950's, and a previous bridge at this site was constructed in the 1920's. The southwest abutment of the existing 4th Street Bridge is located in a cut section along the riverbank with the road elevation at the abutment approximately 20 feet below the ground surface at the top of the bluff to the southwest. The bridge abutment for the previous bridge is located on the riverbank slope north of the existing abutment. Both abutments appear to be in good condition with no apparent distress associated with slope or foundation movement. The top of the block retaining wall of the existing southwest abutment appears to have been disturbed, however, the remainder of the wall generally appears to be in good condition. Surface water discharges from a pipe at the top of the retaining wall beneath the bridge, and has caused moderate erosion down into the bedrock below the bridge.

The riverbank slope northwest of the existing bridge abutment is a very steep slope with a relatively flat surface above the slope. Bedrock exposed in the slope has eroded to a relatively rugged, uneven surface. Based on recent topographic mapping completed for this project, the overall riverbank slope is approximately 1.1h:1v. The upper approximately 20 feet of the slope, where the bedrock appears to be more resistant to erosion, are somewhat steeper. A hiking/biking trail has been constructed along the river channel at the base of the slope. Below the bike path, the river bank slopes gently down to the river, which under low flow conditions, is located along the northeast side of the river bed adjacent to the floodwall. The first existing bridge pier northeast of the abutment is located immediately southwest of the river channel.

The riverbank slope is generally either barren exposed rock, soil, or colluvium. The slope is either barren, covered with grasses, or covered by deciduous trees and grasses. The flatter riverbank adjacent to the river is generally covered by grasses and low brush.

2.2 PROPOSED CONSTRUCTION

We understand current plans for the new bridge call for construction as two separate structures, one for eastbound lanes and one for westbound lanes. The abutment conditions for the two bridge structures differ significantly. The abutment for the westbound structure will be located in a cut section that will be excavated into the bluff, while the abutment for the eastbound structure will be located between the bluff and the existing abutment where a fill section will be required. The maximum height of the fill section for the eastbound lanes is approximately 16 feet. The first pier for both bridge structures will be located approximately 150 feet from the abutment between the hiking/biking trail and the river.

As currently envisioned, the bridge deck will rest on a pedestal with a built-in backwall that will retain approximately 13 feet of geogrid reinforced earth materials, and the road pavement section. The pedestal will be supported by drilled shafts that will extend vertically down into the foundation bedrock. A riprap-covered earth slope will be constructed below the pedestal extending down to the river. We understand that the hiking/biking trail may be relocated part way up onto the riverbank slope to improve access to the trail. The current concept for handling surface water discharge will incorporate a stepped manhole drop structure to be located on the riverbank slope southeast of the existing bridge. The discharge from this structure will be near the edge of the river, downstream of the proposed new bridge and the existing bridge structure.

Construction of the roadway section approach to the southwest abutment will involve a cut on the northwest side and a fill on the southeast side.

3. GEOLOGICAL AND GEOTECHNICAL INVESTIGATIONS

Field and laboratory investigations were completed by Goodson & Associates, Inc. (GAI) to develop final geotechnical engineering criteria and recommendations for final design of the project. GAI's geotechnical investigation program for the project included field and laboratory work intended to provide information for Michael W. West & Associates, Inc. (MWW&AI) to use in evaluating the stability of the southwest abutment, and to characterizing the Headcut Erodibility Index as described in this report. Following is a description of the subsurface exploration program and laboratory testing work that was completed in part to provide information for the engineering evaluations undertaken by MWW&AI for final design of the project.

3.1 SUBSURFACE EXPLORATION PROGRAM

Subsurface investigations relating to the stability of the southwest abutment and riverbed erodibility included: drilling borings BR-1 and BR-2 near the ends of the southwest abutment, and BR-3 near the location of Pier 2 adjacent to the Arkansas River; and excavation of exploratory trench ET-1 adjacent to the bluff, located approximately between the eastbound and westbound lanes of the proposed new roadway. Additional information on ground water levels in the southwest abutment area was obtained from boring DC-1. Details pertaining to the these subsurface investigations are provided in the GAI report.

3.2 LABORATORY TESTING

The laboratory testing program was designed to confirm visual field classification and to provide specific data on the physical, index and engineering properties of the soils and bedrock materials encountered during the subsurface investigation. Laboratory testing included: moisture and density (ASTM D2216 and ASTM D2937); Atterberg limits (ASTM D4318); grain size analysis (ASTM D422, D1140 and D1140); unconfined compressive strength with stress/strain measurement (ASTM D 3148); splitting tensile strength by method of Brazilian disk (ASTM D 3967); and direct shear testing (ASTM D3080 and D5607). Laboratory test results for the specific tests conducted to support the evaluation of the southwest abutment stability and the erodibility index are provided in Attachment A. A summary of laboratory test results for samples obtained from the exploratory borings in the vicinity of the southwest abutment is provided in Table 1.

4. SITE GEOLOGY

Bedrock units located within and near the project site consist of marine sedimentary materials originally deposited during transgression of the Cretaceous Inland Seaway which covered a large portion of what is now Colorado.

Based upon published geologic mapping (Scott, 1964; Scott, 1969; and Scott and Woban, 1986), the proposed southwest abutment for the new 4th Street Bridge is underlain by the late Cretaceous-age (deposited 97.5 to 66.4 million years ago) Lower Transition Member of the Pierre Shale and the Upper Chalk and Upper Chalky Shale units of the Smoky Hill Shale Member of the Niobrara Formation. Bedrock is locally mantled by unconsolidated colluvial and alluvial deposits of Quaternary age (less than 1.6 million years old) as well as some artificial (man-placed) fill. We discuss the stratigraphy and structure of these units below in ascending stratigraphic order.

4.1 BEDROCK STRATIGRAPHY AND STRUCTURE

Scott (1969) describes the Upper Chalky Shale Unit of the Smoky Hill Shale Member of the Niobrara Formation as about 265 feet (thickness) of dark-yellowish-orange, grayish-orange and pale-yellowish-brown soft, fissile shale beds interbedded with dark-gray, hard, platy, chalk. Based on our interpretation of bedrock outcrops and Scott's (1969) description, we believe that the Upper Chalky Shale Unit comprises the near vertically-faced, cliff-forming rocks to the north of the project site along the banks of the Arkansas River.

Bedrock of the Upper Chalky Shale Unit of the Smoky Hill Shale Member is known to contain 17 distinct, measurable bentonite beds, 0.5- to 3.5-inches thick. Scott (1969) asserts that these bentonite beds lie between 180 and 205 feet above the base of the unit, or 60 feet below the base of the overlying Upper Chalk Unit. Abundant sulfates in the form of gypsum are also present within the Upper Chalky Shale Unit.

Scott (1969) describes the Upper Chalk Unit of the Smoky Hill Shale Member, which overlies the Upper Chalky Shale Unit, as 8 feet (thick) of olive-black, blocky, platy to even-bedded chalk. The chalk weathers to dark-yellowish-orange and in fresh outcrops is difficult to distinguish from the underlying and overlying chalk beds. According to Scott (1969), the only foundation problem presented by this unit is the difficulty in excavation.

A fixed line survey was completed for this project as discussed Section 4 below. This survey was completed in what appears to be the Upper Chalk Unit. No exposures of the underlying bedrock units exist within the immediate project area.

Scott (1969) describes the Lower Transition Member of the Pierre Shale, which overlies the Upper Chalk Unit, as 228 feet (thick) of mostly calcareous shale. Scott (1969) further explains that the entire member is generally uniform except for the lower 50 feet which consists of dark-gray chalk beds. The Pierre Shale is also known to contain swelling clays.

Each of the bedrock units described above contain various claystone, shale and chalk strata that are differentiated by geologists using characteristic fossils or fossil imprints. Figure 1 illustrates our interpretation of the site specific geologic relationships at the southwest abutment for the proposed 4th Street Bridge.

Locally the Niobrara Formation and Pierre Shale dip gently 16°± to the east-northeast. Measurements in exploratory trench ET-1 at the southwest abutment indicated the bedding strikes at N32°E to N7°W dipping 2° to 8° to the east. No faults or folds are known to exist within the general project area that would adversely affect the project as currently envisioned.

4.2 SURFICIAL DEPOSITS

Bedrock of the Niobrara Formation and the Pierre Shale is mantled by the Quaternary-age (deposited less than about 1.6 million years ago) Louviers alluvium, Post Piney Creek alluvium, colluvium, and local artificial fill. Scott (1969) describes the Louviers Alluvium within and near the project site as up to 20 feet (thick) of yellowish-brown poorly-stratified, sandy cobbles and gravels with scattered boulders. Specifically, based on our site reconnaissance, the Louviers Alluvium is up to 8 to 10 feet thick and exposed at the top of the present stream bank as illustrated on Figure 1.

We mapped colluvial/residual soils along the base and up into the steeper sections of the abutment slope. This material is generally a sandy, silty clay derived as slope-wash from the bedrock outcrops along the Arkansas River valley, and grades into the lower flood deposits. Flood plain deposits along the river banks are classified as Quaternary-age Post Piney Creek alluvium and are described by Scott (1969) as yellowish-gray, poorly sorted cobbles, gravels, sand and silt. Post Piney Creek alluvium is generally less than about 10 feet thick.

A small berm has been constructed at the base of the river-bank slope, most likely to protect the bike trail from raveling gravels and cobbles originating from the alluvium at the top of the slope. Remnants of the older concrete abutment and other construction materials are also present just north of the existing bridge abutment. We have classified this material as artificial fill and illustrate its general location on Figure 1.

The colluvial/residual soils, Piney Creek alluvium or artificial fill materials may contain bentonitic (swelling) clays derived from local bedrock units.

4.3 STABILITY OF SOUTHWEST ABUTMENT SLOPE AREA

During our site reconnaissance, we observed several slope instability hazards related to the bedrock materials in the southwest abutment area. Based on our field studies, it appears that two near-perpendicular joint sets observed in exposed bedrock units create wedge-type slope failures. These wedge failures and resulting block slides can be seen just north of the proposed 4th Street Bridge abutment site to the northern edge of the mapped area. Where the bedrock is exposed as a near-vertical cliff, these rock slides appear as dislocated, back-tilted blocks. Where, according to the published geologic map, the Upper Chalk Unit and Upper Chalky Shale Unit are exposed beneath the Pierre Shale, these rock slides appear as landslides or as soil talus slopes. A site specific slope stability analysis is presented in Section 7.

5. SUBSURFACE CONDITIONS

Subsurface investigations completed for final design of the southwest abutment included drilling three relatively deep borings, and the excavation of an exploratory trench. The borings provided (1) information related to the stratigraphy in the southwest abutment area, (2) samples for laboratory testing to characterize the physical, index and engineering properties of the subsurface materials, and (3) ground water levels. Exploratory trench ET-1 was excavated to investigate the possibility that a relatively large block within the riverbank slope is sliding toward the Arkansas River. The possibility of a dislodged block in the location of the proposed abutment for the westbound lanes was of concern. The bluff in this location is relatively high and steep in comparison to other portions of the slope along the southwest side of the Arkansas River. Following is a description of the subsurface conditions based on the investigations completed in the southwest abutment area.

5.1 STRATIGRAPHIC PROFILE AND MATERIAL PROPERTIES

In March and May of 2006, Borings BR-1 and BR-2 were drilled near the west and east ends of the southwest abutment, respectively, and Boring BR-3 was drilled near the proposed location of Pier 2 within the Arkansas River channel. Boring logs are provided in the accompanying GAI report. Boring BR-1, drilled from the top of the bluff at the southwest abutment, penetrated 8.5 feet of gravelly sand (Louviers alluvium), over claystone/shale to 31 feet depth where chalky shale was encountered to the bottom of the boring at 135.5 feet depth. Boring BR-2, drilled approximately 21 feet lower in elevation just west of existing Highway 96, penetrated 11.5 feet of fill over claystone/shale that extended down to 22 feet depth where chalky shale was encountered extending to the bottom of the boring at 103.5 feet depth. Boring BR-3, drilled just south of the Arkansas River, penetrated 8 feet of sand (Post Piney Creek alluvium), over chalky shale that extended to the bottom of the boring at 75 feet depth.

The Louviers alluvial sand was fine- to coarse-grained with gravel, and graded to a sandy gravel. Blow counts ranged from 14 blows to drive a California barrel sampler 12 inches (each blow corresponds to a 140 pound hammer dropping 30 inches) to 21 and 22 blows to drive a standard split-spoon sampler 12 inches. Laboratory grain size analysis of one sample showed 56 percent gravel, 28 percent sand, and 16 percent minus No. 200 sieve fines. Cobbles up to 6 inches in size are common at the surface of the cut-slope northwest of the existing westbound lanes, and in the upper portion of exploratory trench ET-1 as described below. The sand/gravel was generally slightly moist to wet, and was typically light brown and gray.

The Post Piney Creek alluvium typically consisted of a clayey sand. Blow counts were 7 and 10 blows to drive a standard split-spoon sampler 12 inches. Grain size analysis of one sample indicated 23.6 percent gravel, 40.1 percent sand and 36.3 percent fines, with a liquid limit of 32, a plasticity index of 18, and a water content of 17 percent.

The fill encountered in the upper 11.5 feet of BR-2 was a clayey sand to sandy clay. The blow counts were 20 and 27 blows to drive a California barrel sampler 12 inches. One grain size analysis indicated 8.7 percent gravel, 56.6 percent sand and 34.7 percent fines. Atterberg limit testing indicated the material is non-plastic. One sample indicated a water content of 2.0 percent and a dry unit weight of 112.3 pounds per cubic foot (pcf). The sand fill was generally slightly moist to wet, and was typically light brown and gray. A swell/consolidation test showed 2.7 percent compression when water was added at an applied pressure of 500 pounds per square foot (psf).

The claystone/shale was slightly sandy to sandy. Blow counts ranged from 22 blows to drive a California barrel sampler 12 inches, to 50 blows to drive a California barrel sampler 6 inches. Laboratory grain size analyses indicated 2.5 to 22.9 percent sand and 77.1 to 97.5 percent fines. Liquid limits ranged from 43 to 52, and plasticity indices ranged from 20 to 25, while water contents ranged from 11.9 to 14.0 percent and dry unit weights ranged from 97.0

to 106.9 pcf. The claystone/shale was typically slightly moist to moist and was orange-brown, and light brown to dark brown. A swell/consolidation test on the claystone/shale showed 0.2 percent swell when water was added at an applied pressure of 500 psf. A direct shear test on the claystone/shale indicated a peak shear strength corresponding to a cohesion of approximately 90 psf and a friction angle of 48 degrees, and an ultimate shear strength corresponding to a cohesion of approximately 550 psf and a friction angle of 35 degrees.

The chalky shale was slightly weathered to unweathered, strong to very strong rock. Rock Quality Designation (RQD) values ranged from 33 to 96 in the upper 10 to 15 feet cored, and from 85 to 100 in the deeper shale bedrock. Fractures were generally narrow to very narrow with no filling, planar, and horizontal to nearly horizontal. Grain size analyses of the shale indicated 26.9 to 83.1 percent fines. Atterberg limit testing indicated one sample was non-plastic with two others showing liquid limit values of 26.7 and 51.3, and plasticity indices of 12.2 and 35.3. Water contents ranged from 1.0 to 1.9 percent and dry unit weights from 139.4 to 145.9 pcf. The shale bedrock was generally light gray to dark gray with whitish speckles of calcium carbonate. Bentonite seams 1/16-inch to 2-inch thick were found every approximately 5 to 10 feet throughout the majority of the shale that was cored in borings BR-1, BR-2 and BR-3. Direct shear testing on saw cut surfaces in the shale indicated ultimate shear strengths corresponding to cohesion values of approximately 240 and 270 psf, and friction angles of 24 and 31 degrees. A direct shear test on a saw cut surface through a bentonite bed indicated an ultimate shear strength corresponding to a cohesion of approximately 60 psf and a friction angle of 23 degrees. The unconfined compressive strength of shale test specimens ranged from 1,050 to 12,580 pounds per square inch.

Piezometers were installed in borings BR-1, BR-2 and BR-3 to monitor ground water levels. Water levels were measures at depths of approximately 38 to 41 feet in BR-1, 26 to 27 feet in BR-2, and zero to 2.5 feet in BR-3.

Boring logs and complete laboratory test results for borings BR-1, BR-2 and BR-3, and all of the other borings completed for the final design investigation are included in the accompanying GAI report. Results of laboratory test results specifically completed for this study are summarized on Table 1 and are included in Attachment A.

5.2 EXPLORATORY TRENCH ET-1

On May 23 and 24, 2006, exploratory trench ET-1 was excavated along the slope immediately east of the billboards that are located on the bluff at the southwest abutment of the proposed 4th Street Bridge. The total length of the trench was approximately 120 feet. Due to benching that was required to construct a stable trench, two exposures of the claystone/shale bedrock were excavated, logged and photographed. The location of the trench is shown on Figure 1, and logs of the lower and upper trench faces are provided on Figures 2 and 3, respectively.

The upper approximately 8 to 10 feet of the west side of the trench slope consisted of sand and gravel with cobbles up to approximately 6 inches in size. Beneath the sand and gravel, we found claystone/shale bedrock that was closely to very closely fractured, moderately weathered to weathered, with gypsum crystals common in the fractures.

As shown on Figures 2 and 3, six bentonite beds were mapped across the bedrock exposures in exploratory trench ET-1. Where exposed in the exploratory trench, the bentonite beds were continuous and did not show vertical displacement. Based on observations made in the exploratory trench and in the southwest abutment bluff area, we believe the bedrock has not been displaced by block movement within approximately 120 feet of the southwest slope of the Arkansas River valley at the proposed bridge location.

6. RIVERBED ERODIBILITY INDEX

In order to evaluate scour potential and to assist with design of the bridge pier to be constructed in the Arkansas River channel, we estimated the Headcut Erodibility Index following the procedures outlined in Chapter 52, Field Procedures Guide for the Headcut Erodibility Index (Natural Resource Conservation Service, 1997). Our scope of work included a site-specific engineering geological investigation to assist the design team in determining the Headcut Erodibility Index for the riverbed bedrock material at the location of proposed Pier 2. We discuss the details of our field investigation and calculation of the Headcut Erodibility Index in the following paragraphs.

6.1 FIELD INVESTIGATION

Information obtained from the field investigations that was used to estimate the Headcut Erodibility Index included: (1) a fixed line survey along an exposure of bedrock located beneath the previous 4th Street Bridge abutment on the southwest bank of the Arkansas River; (2) RQD information and joint roughness observation of rock core obtained from boring BR-3; and (3) measurements of attitudes of fractures, joints and bedding exposed in exploratory trench ET-1. For the fixed line survey we measured and recorded the attitudes of fractures/joints or bedding (discontinuities) that crossed a 10 foot length of surveying tape following the standardized procedure detailed in Chapter 52, Field Procedures Guide for the Headcut Erodibility Index (NRCS, 1997). We provide the results of our fixed line survey in Table 2.

6.2 ESTIMATE OF THE HEADCUT ERODIBILITY INDEX

The Headcut Erodibility Index (NRCS, 1997) represents a measure of the resistance of a particular earth material, rock or soil, to erosion. The Headcut Erodibility Index, K_h, is the product of four individual parameters as follows:

$$K_h = M_s \times K_b \times K_d \times J_s$$

where for a rock material:

M_s = material strength number;
K_b = block size number;
K_d = discontinuity shear strength number; and
J_s = relative ground structure number.

We have estimated values for each of the four parameters based on information and interpretations of information collected during the field and laboratory investigations. Typically we have estimated a low, high and most likely value for each parameter. The estimate of the headcut erodibility index is summarized on Table 3. Following is a brief description of each parameter and the values we selected.

The material strength number, M_s, is an empirically derived value based on the uniaxial compressive strength of the bedrock. Based on results of uniaxial compressive strength testing of samples obtained from boring BR-3, and Table 52-4 (NRCS, 1997), we estimate low, high and most likely values of 8.5, 12.5 and 12.5, respectively.

The block size number, K_b, is a representation of the mean size of rock masses between discontinuities (i.e. joints, fractures, bedding), and is based primarily on the Rock Quality Designation (RQD). The RQD is defined as the cumulative length of core pieces longer than 10 cm in a run divided by the total length of the core run. The RQD value used to obtain the block size number was obtained from the first three 5-foot long core runs that were completed in boring BR-3. The average RQD value for these three runs was 77. A J_n value of 5 was obtained from Table 52-5 (NRCS, 1997), based on the presence of more than four joint sets. The low, high and most likely values of the block size number, K_b, were all taken as 15.5.

The discontinuity shear strength number, K_d, is another empirical value based on interpretation of observed rock core materials in this case. The discontinuity shear strength number is the quotient of the joint roughness number, J_r, and the joint alteration number, J_a. Based on observation of rock core from boring BR-3 and Tables 52-8 and 52-9 (NRCS, 1997), we selected a value of $J_r = 1.5$ assuming rough/irregular planar surfaces, and a value of $J_a = 1.0$ assuming narrow, clean open joints with no infilling. Based on these parameters, we have used a low, high and most likely discontinuity shear strength number, K_d, of 1.5.

The relative ground structure number, J_s, is an empirical parameter based on the attitude of the least favorable discontinuity (including bedding) with respect to the direction of flow of water, in this instance the direction of flow of the Arkansas River at the proposed pier location. The low, high and most likely values determined for the relative ground structure number, J_s, are 0.45, 0.52 and 0.52, respectively.

Using the values discussed above, the Headcut Erodibility Index for the near surface shale bedrock in the vicinity of Pier 2 within the Arkansas River channel is estimated to range from approximately 90 to 150. These values are based on observations at one relatively small (11 feet long and 4 feet tall) bedrock exposure located beneath the existing 4th Street Bridge, and observation of the rock core obtained from boring BR-3 near the proposed pier location. Variations can and do occur in geological materials and departures from conditions presented in this report are possible.

7. SOUTHWEST ABUTMENT SLOPE STABILITY EVALUATION

The existing riverbank/abutment slope on the southwest side of the Arkansas River at the proposed southwest abutment location slopes up to approximately 41°± (1.15h:1v) based on project topographic mapping. The slope is comprised of weathered shale and chalk, locally covered by colluvial soils. In addition, several rock slides and landslides were observed on the same slope further upstream within the general project area, originating in the same bedrock materials that comprise the southwest abutment of the proposed 4th Street Bridge. Given these site conditions, and considering the presence of bentonite beds within the shale bedrock, a slope stability evaluation of the southwest abutment was completed as part of this study. As described in Section 5.2, based on observations made in the exploratory trench and in the southwest abutment bluff area, we believe the bedrock has not been displaced by block movement within approximately 120 feet of the southwest slope of the Arkansas River valley at the proposed bridge location. The slope stability analyses presented in this section were conducted to evaluate the stability of the proposed abutment slope and to help develop parameters for the design of drilled shafts that will support the abutment seat.

7.1 GENERAL

Slope stability analysis is a means of assessing the relative stability of a slope and increases our understanding of the factors that contribute to stability or instability of a slope. All stability analyses require certain basic data that must either be gathered in the field or be assumed by the analyst. These data include: (1) a topographic profile (study section) representative of the subject slope; (2) material properties including the shear strength of representative soil and bedrock materials, typically stated in terms of cohesion and angle of internal friction; (3) depth to the phreatic surface and fluctuations of that surface with time, and; (4) geometry of the failure surface or assumption of a standard geometry, i.e., circular or non-circular. A prudent and meaningful slope stability model must consider all of these factors and the uncertainties inherent in each.

For our analysis, we used UTEXAS4, a two-dimensional limit equilibrium modeling software developed at the University of Texas (Wright, 1999). We used shear strength and soil index property data developed from our test holes and laboratory test results, as well as interpretive input based on our understanding of site geologic conditions. We discuss the input and assumptions made for the analyses, and results of our slope stability calculations below.

7.2 ELEMENTS OF SLOPE STABILITY ANALYSES

7.2.1 Study Section

Our slope stability study section passes through the proposed southwest abutment of the westbound structure as shown on Figure 1. The geometry of the slope at the study section represents the post-construction configuration where the top of the bluff has been excavated down to the level of the westbound lanes, and the slope below the southwest bridge abutment has been constructed at 2h:1v (horizontal:vertical).

Exploratory borings and exploratory trench ET-1 located in the vicinity of the study section provided subsurface information used to develop the stratigraphy within the slope. Based on the subsurface information developed for this study, our interpretation of the geologic conditions, and the proposed construction, we modeled the slope using the following four materials.

- Shale
- Colluvial and Alluvial Deposits
- Bentonite Beds
- Riprap

We modeled bentonite beds at approximately 5 to 10 foot vertical intervals within the shale materials.

7.2.2 Material Properties

Material properties used in the slope stability analyses are summarized on each slope stability analysis figure. A description of the basis for selection of these material properties follows:

Shale. Direct shear strength testing of the claystone/shale bedrock materials is discussed in Section 5.1, and results are summarized in Table 1. Based on these test results, we modeled the claystone/shale materials with a cohesion of 100 pounds per square foot (psf), a friction angle of 44 degrees, and a total unit weight of 145 pounds per cubic foot (pcf).

Colluvial and Alluvial Deposits. The colluvial and alluvial soils are present at the base of the slope and extending out into the Arkansas River channel. Considering that these materials are generally below the phreatic surface and will be saturated, we assumed zero cohesion and a friction angle of 35 degrees, with a total unit weight of 122 pcf.

Bentonite Beds. The bentonite beds represent potential weak planes within the claystone/shale materials. Considering the possibility that these beds have a low shear strength, and the dip of these beds toward the Arkansas River, we conducted direct shear testing on a bentonite bed test specimen obtained from boring BR-2 at 75.5 to 76.2 feet depth. Based on the results of this test we assumed a cohesion of 50 psf and a friction angle of 23 degrees for the bentoninte beds. This represents the strength of the bentonite beds that have experienced shear movement, and is therefore conservative. The total unit weight assumed for the bentonite was 126 pcf.

Riprap. A two foot thick layer of riprap material will cover the slope beneath the southwest bridge abutment. For the purpose of these slope stability analyses, we assumed zero cohesion, a friction angle of 42 degrees, and a total unit weight of 140 pcf for the riprap material.

7.2.3 Ground Water Levels

Our understanding of ground water conditions within the subject slope is based primarily on piezometer water level readings obtained from the borings completed for the geotechnical study. For the slope stability analyses, we assumed a phreatic surface that is approximately 5 feet higher than that indicated by the piezometer water levels. The higher phreatic surface was used to account for potential increases in the water level associated with seasonal fluctuations, and the possibility that the ground water level will rise due to increased development and landscape irrigation in the vicinity of the southwest abutment.

7.2.4 Geometry of Trial Shear Surfaces

The types of slope movement most likely to occur at the southwest abutment include block or non-circular slides with shearing occurring along the bentonite beds, and classical circular-shaped slope failures. The primary objective of the slope stability analyses was to evaluate potential slope movements associated with shearing along discrete bentonite beds, as well as potential circular shear surfaces that pass through the abutment slope. For the non-circular cases analyzed, we examined the stability of the slope for several potential block failures along discrete bentoninte beds located within the shale. For these cases we considered blocks, where shearing would occur along the bentonite beds starting at the point where the drilled shafts, which will support the southwest abutment seat penetrate the beds, then extending out to the face of the slope along the bentonite bed. For the potential circular shear surfaces, we conducted a rigorous search to locate the shear surface that yields the lowest factor of safety, referred to as the critical shear surface.

7.3 RESULTS OF SLOPE STABILITY ANALYSIS

Following is a summary of the results of the slope stability analyses completed for this study. The results are presented graphically and include the factor of safety calculated for each case analyzed. The resultant factor of safety in our limit equilibrium slope stability analyses is determined with respect to shear strength. The factor of safety is defined as the ratio of the available shear strength of the materials in the slope to the shear strength required for "just-stable" equilibrium or the limit equilibrium of the slope. A factor of safety of 1.0 implies that the slope is just stable or in a state of limit equilibrium. A factor of safety greater than 1.0 implies that the slope is stable. Considering that there is typically a significant amount of uncertainty involved in slope stability analysis, factors of safety of 1.3 to 1.5 or greater are often used as the acceptance criteria when evaluating slopes of this type.

We performed a series of slope stability calculations for potential block-type, noncircular, shear surfaces where sliding occurs along the base of the block on the relatively low strength bentonite beds. Results of the slope stability analyses for noncircular shear surfaces where shearing occurs along bentonite beds at varying depth within the shale are provided on Figures 4 through 11. The results for the case where shearing occurs along a shallow bentonite bed is provided on Figure 4 where the factor of safety is found to be 1.7. For the cases where shearing occurs along the next six deeper bentonite beds (Figures 5 through 10), the resulting factors of safety range from 1.9 to 2.1. Where the noncircular shear surface passes through the deepest bentonite bed considered (Figure 11), the factor of safety increased to 2.2. The results for the noncircular shear surfaces presented on Figures 4 through 11 are conservative considering: (1) that the ultimate strength of the bentonite beds as indicated by the laboratory testing was used for the slope stability calculations; and (2) that the shearing resistance of the drilled shafts was not incorporated in these calculations.

Results of the noncircular shear surface cases were also used to evaluate the lateral resistance available where the drilled shafts deflect laterally toward the river. Results of this evaluation, along with the lateral deformation analysis of the drilled shafts performed using properties for subsurface materials provided in the GAI geotechnical report, provided the basis for design of the drilled shafts at the southwest abutment.

Additional slope stability analyses were conducted considering circular shear surfaces passing through the southwest abutment slope. Figure 12 illustrates the critical circular shear surface for the southwest abutment slope, which has a factor of safety of 1.8. For this case a search routine within the slope stability program was used to locate the potential shear surface that yielded the lowest factor of safety. As shown on Figure 12, the critical circular shear surface is located in the lower portion of the slope, passing predominantly through the colluvial and alluvial deposits. Figure 13 presents the result for a circular shear surface that passes through the full height of the slope, yielding a factor of safety of 2.1. The factors of safety calculated for circular shear surfaces are in general agreement with the results for noncircular shear surfaces as discussed above.

7.4 DISCUSSION

Slope stability analyses conducted to evaluate the southwest abutment slope for the proposed 4th Street Bridge indicate that, in our opinion, the slope should have an adequate margin of safety. Estimation of the factor of safety for noncircular shear surfaces indicate factors of safety of 1.7 to 2.1, while results for circular shear surfaces indicate factors of safety in the range of 1.8 to 2.1.

Results of slope stability analyses presented in this report assume somewhat conservative phreatic conditions in the southwest abutment slope. However, if significant irrigation or ponding of water occurs in the vicinity of the southwest abutment, the ground water levels could rise above those considered in these analyses. Such increases in the ground water level will reduce the shear strength of the bentonite beds, and will decrease the relative stability of the slope. Should plans involve significant modification of surface water management in the vicinity of the southwest abutment, additional slope stability analyses may be warranted to evaluate the impact on the stability of the abutment slope.

8. DESIGN RECOMMENDATIONS

The design recommendations presented in this section are intended for use in the Final Design Phase (FOR) for the project. These recommendations are based on our understanding of the project, the available subsurface information, literature reviewed, the results of field and laboratory investigations, and the engineering analyses conducted for this study. Additional geotechnical design and construction criteria and recommendations are provided in the accompanying report prepared by GAI.

8.1 SOUTHWEST ABUTMENT SLOPE BACKFILL

We understand that the previous and existing bridge abutment structures will be removed as part of the construction of the new bridge. Construction of the 2h:1v slope beneath the new bridge at the southwest abutment and across the adjacent slopes will require placement of embankment fill where the bridge abutments are removed and where the slope needs to be raised to the design grade. Prior to placement of embankment fill, any loose material on the slope should be removed and the exposed surface benched and prepared in accordance with CDOT requirements. We recommend that the material used as slope fill be a clayey or silty sand with moderate cohesive strength such that the outer portion of the slope will not ravel when placed in accordance with CDOT specifications.

8.2 CUT SLOPE

Current plans indicate that the abutment bearing seat at the southwest abutment will be supported on eight 48-inch diameter drilled shafts. The portion of the abutment bearing seat that will support the bridge deck for westbound lanes will be located in a cut section in the bluff northwest of the existing abutment. The abutment bearing seat that will support the bridge deck for the eastbound lanes will be constructed on a fill section that will be approximately 16 feet above the existing ground surface in this area. The base of the excavation for the westbound lanes will be approximately 11 feet below the top of the bluff. We estimate that about 8 to 10 feet (thickness) of alluvial sands and gravels overly claystone/shale bedrock in the bluff area. The excavation will likely expose sands and gravels over claystone/shale bedrock.

Construction of the previous bridge involved excavation into the bluff in the area west of the existing bridge's south-west abutment. Topographic mapping completed for this project indicates that the height of the existing cut slope that extends along the west side of what appears to be the cut made for the previous bridge, ranges up to approximately 20 feet, with the highest section of the cut slope located near where the cut slope intersects the riverbank slope. The steepness of this existing cut slope varies from about 1.7h:1v to 2.2h:1v. Weathering and erosion of the slope has created some loose material on the surface of the slope and at the base of the slope.

The proposed excavated slope that will be located along the west side of the proposed westbound lanes should be stable at 2h:1v or flatter. Unless slope protection is provided, erosion of the proposed cut slope will likely be similar to the erosion that has occurred on the existing cut slope. Erosion of the existing slope includes relatively minor erosion rills and deposition of the granular materials at the base of the slope, and appears to have essentially resulted in a flatter slope. If the slope needs to be higher than approximately 20 feet, a flatter slope should be considered. The upper portion of the proposed 11 foot cut slope will likely be sand and gravel with cobbles, and the lower portion of the slope will be in the claystone/shale. It appears that the sands and gravels are relatively clean with little cohesion such that the surface of the slope will tend to be loose and will ravel. In addition, the slope will likely need to be covered with a suitable soil material to support grass or other vegetation. Other options for erosion protection may be considered.

9. CONCLUSIONS

1. The proposed southwest abutment for the new 4th Street Bridge is underlain by the late Cretaceous-age (deposited 97.5 to 66.4 million years ago) Lower Transition Member of the Pierre Shale and the Upper Chalk and Upper Chalky Shale units of the Smoky Hill Shale Member of the Niobrara Formation. Bedrock is locally mantled by unconsolidated colluvial and alluvial deposits of Quaternary age as well as some artificial (man-placed) fill.

2. Boring BR-1, drilled on the west side of the southwest abutment penetrated 8.5 feet of gravelly sand (Louviers alluvium), over claystone/shale to 31 feet depth where chalky shale was encountered extending to the bottom of the boring at 135.5 feet depth. Boring BR-2, drilled on the east side of the southwest abutment, penetrated 11.5 feet of fill over claystone/shale that extended down to 22 feet depth where chalky shale was encountered extending to the bottom of the boring at 103.5 feet depth. Boring BR-3, drilled near the proposed location of Pier 2 in the Arkansas River channel, penetrated 8 feet of sand (Post Piney Creek alluvium), over chalky shale that extended to the bottom of the boring at 75 feet depth.

3. Water levels were measured at depths of approximately 38 to 41 feet in BR-1, 26 to 27 feet in BR-2, and zero to 2.5 feet in BR-3.

4. Based on observations made in the exploratory trench and in the southwest abutment bluff area, we believe the bedrock has not been displaced by block movement within approximately 120 feet of the southwest slope of the Arkansas River valley at the proposed bridge location.

5. The Headcut Erodibility Index for the near surface shale bedrock in the vicinity of Pier 2 within the Arkansas River channel is estimated to range from approximately 90 to 150.

6. Slope stability analyses conducted to evaluate the southwest abutment slope for the proposed 4th Street Bridge indicate that the slope, in our opinion, should have an adequate margin of safety. Estimation of the factor of safety for noncircular shear surfaces indicate factors of safety of 1.7 to 2.1, while results for circular shear surfaces indicate factors of safety in the range of 1.8 to 2.1.

7. Results of slope stability analyses presented in this report assume somewhat conservative phreatic conditions in the southwest abutment slope. However, if significant irrigation or ponding of water occurs in the vicinity of the southwest abutment, the ground water levels could rise above those considered in these analyses. Such increases in the ground water level will reduce the shear strength of the bentonite beds, and will decrease the relative stability of the slope. Should plans involve significant modification of surface water management in the vicinity of the southwest abutment, additional slope stability analyses may be warranted to evaluate the impact on the stability of the abutment slope.

8. Construction of the 2h:1v slope beneath the new bridge at the southwest abutment and across the adjacent slopes will require placement of embankment fill where the bridge abutments are removed and where the slope needs to be raised to the design grade. Prior to placement of embankment fill, any loose material on the slope should be removed and the exposed surface benched and prepared in accordance with standard CDOT requirements. We recommend that the material used as slope fill be a clayey or silty sand with moderate cohesive strength such that the outer portion of the slope will not ravel when placed in accordance with CDOT specifications.

9. The proposed cut slope along the west side of the westbound lanes at the southwest abutment should be stable at 2h:1v or flatter. Unless slope protection is provided, erosion of the cut slope will likely be similar to the erosion that has occurred on the existing cut slope. If the slope needs to be higher than approximately 20 feet, a flatter slope should be considered.

10. GENERAL INFORMATION

Information in this report is intended to provide an assessment of geological conditions and slope stability for the southwest abutment of the proposed bridge. The report is based on review of existing information, our geological studies and site-specific investigations, our general understanding of geological processes in the project area, our understanding of the proposed construction, and our past experience with similar conditions. Variations can and do occur in geological and constructed materials and departures from the conditions portrayed in this report are possible. The performance of the project cannot be guaranteed in any respect. No other use is intended or implied by the information, opinions, conclusions and recommendations contained herein.

If you have any questions regarding this report, please do not hesitate to call.

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TABLES

TABLE 1
SUMMARY OF LABORATORY TEST RESULTS
4st STREET BRIDGE
SOUTHWEST ABUTMENT EVALUATION

				Atterberg Limits			Grain-Size Distribution					Direct Shear Strength						
												Peak Shear Strength		Ultimate Shear Strength				
Hole	Depth (ft)	Water Content (%)	Dry Unit Weight (pcf)	Liquid Limit	Plastic Limit	Plasticity Index	Gravel Sizes (%)	Sand Sizes (%)	Minus No. 200 Sieve Fines (%)	Clay Fines (%)	USCS	Cohesion (psf)	Friction Angle (degrees)	Cohesion (psf)	Friction Angle (degrees)	Swell (+) / Consol (-) at 500 psf Applied Pressure (%)	Tensile Strength (psf)	Unconfined Compressive Strength (psi)
BR-1	0-5						55.7	27.9	16.4		GM							
BR-1	10	13.9	102	52	27	25	0	22.9	77.1		CH					0.2		
BR-1	15	13.3	106.9															
BR-1	15	14	97	43	23.2	19.7	0	2.5	97.5		CL	89.3	47.9	553.0	34.8			
BR-1	15	11.9	106.8															
BR-1	41.0-42.0																	8,850.0
BR-1	42.0-42.2					NP	0	16.9	83.1	10.7								
BR-1	47.0-48.5													275.5	31.1			11,250.0
BR-1	65.5-66.4	1.6	145.9															2,540.0
BR-1	100.0-101.9																	3,890.0
BR-2	5	2	112.3			NP	8.7	56.6	34.7		SM					-2.7		
BR-2	10																	
BR-2	30.2-32																	10,860.0
BR-2	38.5-39.5			26.7	14.6	12.2	0	73.1	26.9					240.1	24.5			
BR-2	45.7-47.2																	1,670.0
BR-2	55.6-57.3	1.9	139.4															2,590.0
BR-2	69.8-71.4																	12,580.0
BR-2	75.5-76.2			51.3	15.4	35.3	0	18.1	81.9	8.6				59.2	23.0			
BR-3	5	17		32	14	18	23.6	40.1	36.3		SC							
BR-3	11.1-11.8																240	1,250.0
BR-3	24.0-25.0																690	1,050.0
BR-3	32.8-33.6	1	143.4															2,350.0

NOTES:
1. Laboratory testing conducted by Advanced Terra Testing, Inc.
2. Percent fines test result for BR-2 at 38.5-39.5 feet appears to be low likely due to difficulty breaking down shale rock fragments.
3. L/D < 2.0 correction factor applied to Unconfined Compressive Strength for BR-3 at 11.1 to 11.8 feet depth.

TABLE 2
RESULTS OF FIXED LINE SURVEY

STATION (FT.-IN.)	STRIKE	DIP	JOINT ROUGHNNESS	JOINT FILL	JOINT SIDEWALL ALTERATION	JOINT WIDTH (IN.)	Joint Set
2'-1"	N27°E	80°SE	smooth/undulating	none	none	<1/16"	1
3'-2"	N20°W	45°SW	smooth/undulating	residual gypsum	none	<1/16"	2
5'-2"	N20°W	80°SW	smooth/undulating	loose, clayey	very minor discoloration	1/16"	2
7'-8.5"	N13°E	51°SE	stepped	tight clay	none	1/16" - 1/8"	1
8'-2"	N37°W	71°SW	smooth/undulating	loose, clayey	1/16" - 1/8"	1/8"	2
9'-2"	N43°E	74°SE	smooth/undulating	sandy clay	minor discoloration	1/4" - 1/2"	1

NOTES:

1. Joint sets were determined by natural groupings on a streonet plot.
2. The fixed line survey was located along a ten foot length of exposed Upper Chalk Unit of the Smokey Hill Shale Member of the Niobrara Formation beneath the abutment for the existing 4th Street Bridge.
3. The orientation of the fixed line survey was N65°W approximately parallel to the direction of flow of the Arkansas River at the fixed line survey location.
4. The attitude of bedding along the fixed line survey was N6°W dipping 16°NE.
5. Average bedding frequency/thickness equals 13 bedding planes per inch at the fixed line survey based on measurements taken above the line, below the line and at the line.

TABLE 3
ESTIMATE OF HEADCUT ERODIBILITY INDEX

The Headcut Erodibility Index takes the general form of:

		Low	High	Most likely
$K_h = M_s * K_b * K_d * J_s$	**ESTIMATE OF HEADCUT ERODIBILITY INDEX** Kh =	88.7	150.8	**150.8**

Where:

M_s is the materail strength number of the earth material
K_b is the bolck or particle size number
K_d is the discontinuity or interparticle bond shear strength number
J_s is the relative ground structure number

M_s

Uniaxial compressive strength test results:

	Low	High	Most likely
From Table 52-4, select M_s value of	8.5	12.5	12.5

K_b

$K_b = RQD/J_n =$	15.5	15.5	15.5

Where:

Depth	RQD (BR-3)
10-15	87
15-20	60
20-25	85
25-30	92
30-35	92
35-40	95

Use average of upper three intervals => RQD = 77

J_n from Table 52-5 assuming more than four joint sets = 5.0

K_d

	Low	High	Most likely
$K_d = J_r/J_a =$	1.50	1.5	1.5

Where:

J_r = joint roughness number
J_a = joint alteration number

J_r from Table 52-8, assuming rough/irregular; planar surfaces = 1.5
Ja from Table 52-9, assuming narrow clean, open joint, no infilling = 1.0

J_s

		Low	High	Most likely
Determined according to Table 52-12 See calculations in table below	J_s=	0.45	0.52	0.52

Main Discontinuity Sets (determined from trench (2006) and stringline (2003):

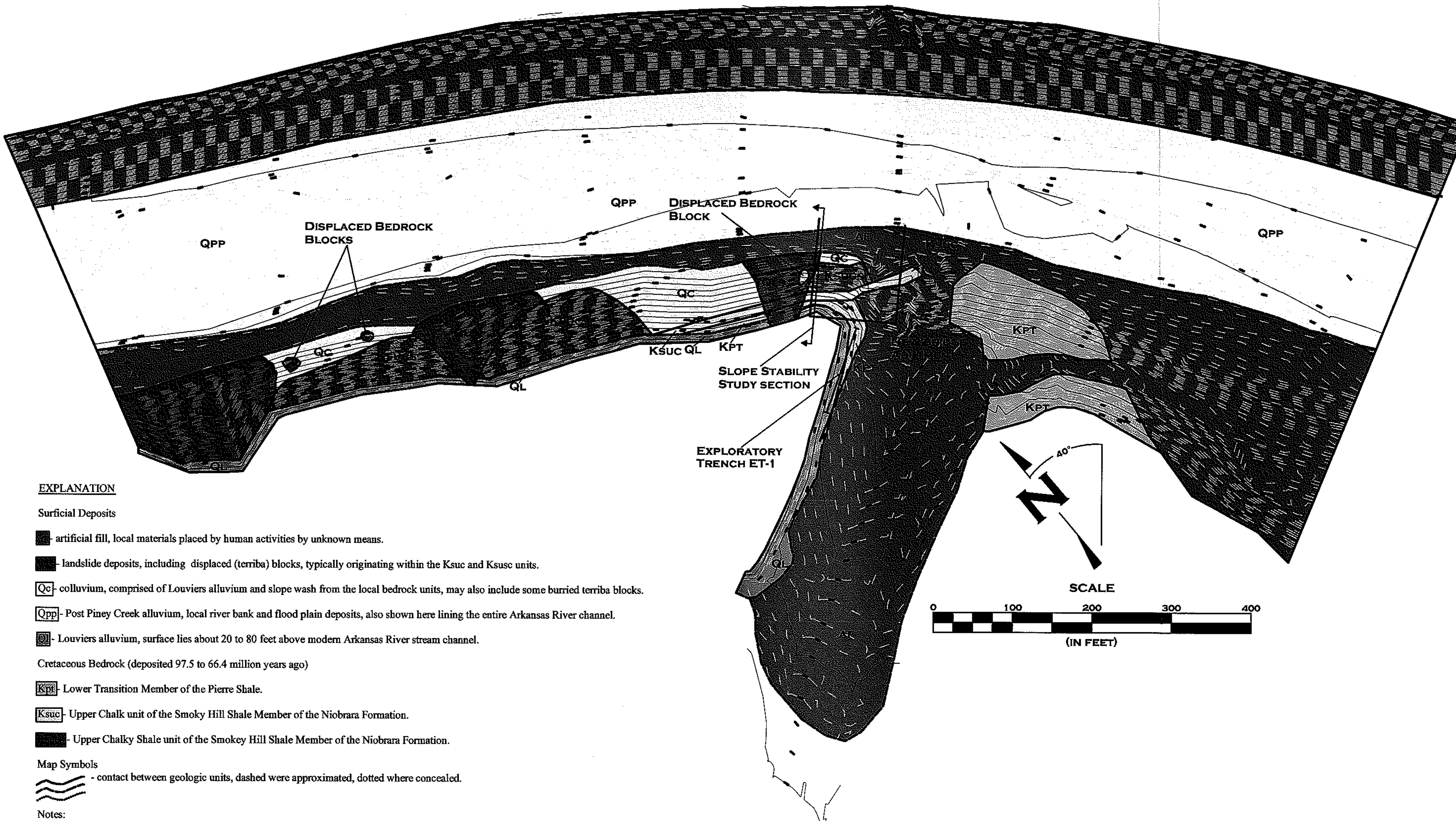
Type	*Dip Direction*	*True Dip Angle*	*Strike*	*Flow Direction*	***tan a** = (tan b)(sin c) [52-11]*	***Apparent Dip (a)***	*Dip Direction - River Flow Direction [p. 52-13]*	***Effective Dip (q)*** *With flow direction => q=a-a [52-12] Against flow direction => q=a+a [52-13]*	**Js Table [52-12] r=1:1**	**Js Table [52-12] r=1:2**
Bedding	109	6	19	132	-0.10	-5.5	337 => with direction of flow	4.9	1.33	1.2
Joint Set 1	182	83	92	132	-5.24	-79.2	50 => with direction of flow	78.6	0.52-0.68	0.45-0.57
Joint Set 2	128	79	38	132	-5.13	-79.0	356 => with direction of flow	78.4	0.52-0.68	0.45-0.57
Joint Set 3	230	69	140	132	0.36	19.9	98 => against direction of flow	19.6	0.52-0.63	0.57-0.7
Joint Set 4	270	88	180	132	21.28	87.3	138 => against direction of flow	87.9	1-1.33	1-1.39
									Maximum=0.52	*Minimum=0.45*

a=apparent dip of discontinuity
b=true dip of discontinuity
c=(strike of discontinuity)-(spillway flow direction)

a=slope of river channel => 0.57

r=ratio of joint spacing

FIGURES



EXPLANATION

Surficial Deposits

- artificial fill, local materials placed by human activities by unknown means.

- landslide deposits, including displaced (terriba) blocks, typically originating within the Ksuc and Ksusc units.

Qc - colluvium, comprised of Louviers alluvium and slope wash from the local bedrock units, may also include some burried terriba blocks.

Qpp - Post Piney Creek alluvium, local river bank and flood plain deposits, also shown here lining the entire Arkansas River channel.

Ql - Louviers alluvium, surface lies about 20 to 80 feet above modern Arkansas River stream channel.

Cretaceous Bedrock (deposited 97.5 to 66.4 million years ago)

Kpt - Lower Transition Member of the Pierre Shale.

Ksuc - Upper Chalk unit of the Smoky Hill Shale Member of the Niobrara Formation.

- Upper Chalky Shale unit of the Smokey Hill Shale Member of the Niobrara Formation.

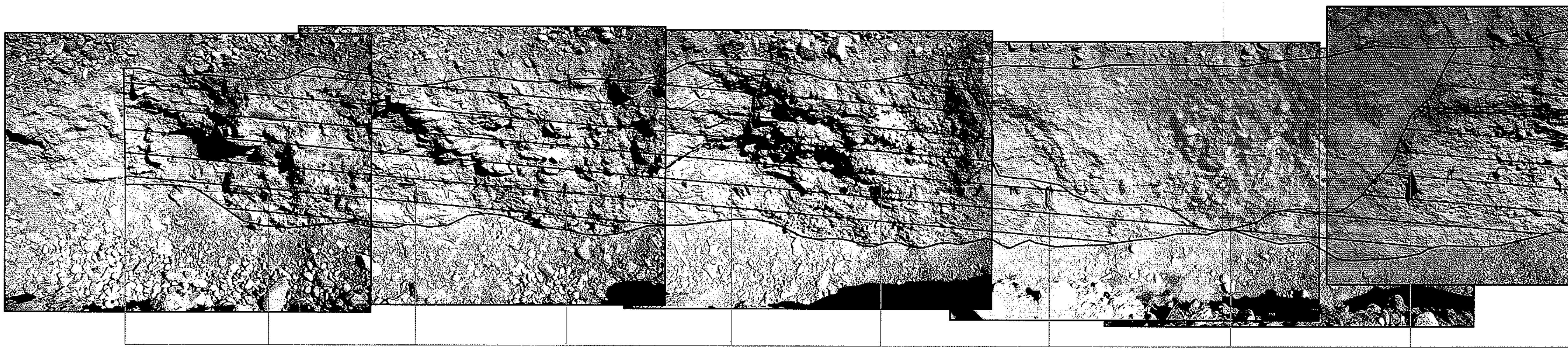
Map Symbols

- contact between geologic units, dashed were approximated, dotted where concealed.

Notes:

1. Geologic field studies conducted on March 17 and April 2, 2003.
2. Base map topography provided by Abel Engineering Professionals, Inc.

MICHAEL W. WEST & ASSOCIATES, INC. 6970 S. Holly Circle, Suite 206 Centennial, Colorado 80112-1066	Geologic Map 4th Street Bridge Pueblo, Colorado	BY: EJM PROJ: 03540	4/28/2003 FIGURE: 1



LEGEND:

Louviers Alluvium (Pleistocene): GRAVELS, cobbly (10-20% cobbles, 30-50% gravels), sandy, silty, tan, well-sorted, rounded to sub-rounded, mostly crystalline origin, some vulcanic and sedimentary rocks, deposit is layered in upper slope and parts of lower portion of trench, and shows more chaotic texture at the bottom of the lower trench.

Shale (Pierre Shale Transition Member or Niobrara Formation - Upper Cretaceous): Calcareous SHALE, thinnly bedded with wavy to plain bedding planes, calcium carbonate speckles are visible in fresh breaks, very closely to closely fractured, breaks apart easily, moderately weathered to weathered with discoloration of rock along fracture surfaces, weak to medium strong, layers of lense-shaped strong, hard and unweathered concretions (about cobble-size) common, gypsum along fractures common. At the contact with Louviers alluvium, the shale is more weathered, nearly clay-like, with no concretions. ***Bedding: ~N7°W/6°E***

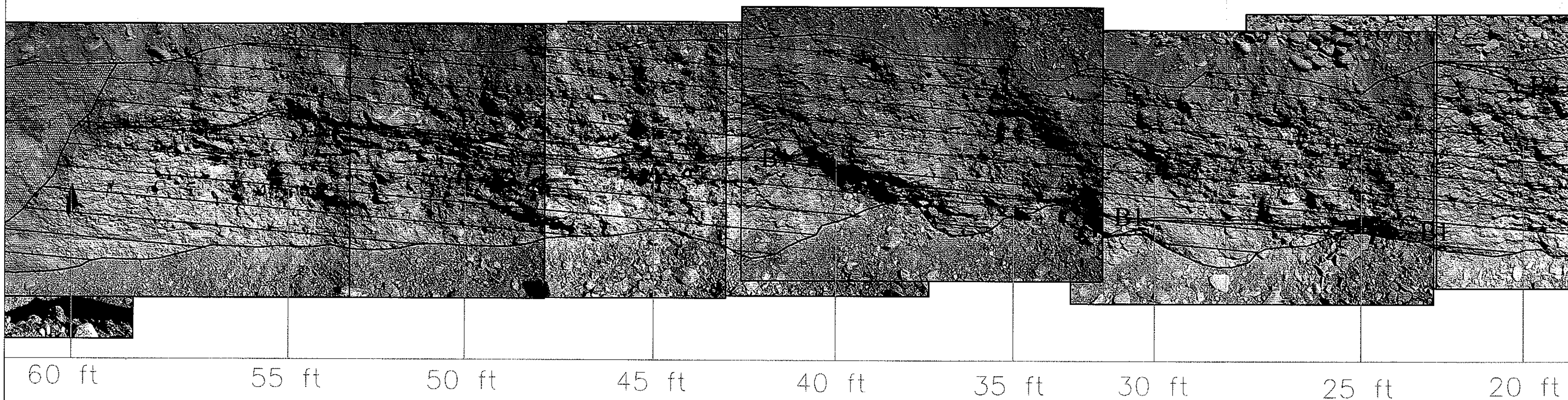
B1 ***Bentonite Beds within Shale:*** Orange-brown in color, weathered.

B1: About ¾" thick; N30°E/5°E, N26°E/10°E, and N5 °E/7°E
B2: About ⅛" thick
B3: About ¼" to ¾" thick; N2°E/2°E to N25°E/8°E
B4: About 1 to 1½" thick
B5: About ¾" thick
B6: About 1" thick; N32°E/8°E

Jointing and Fracturing of the Rock: The shale is typically intensely fractured. Very closely to closely spaced fracturing was observed parallel and sub-parallel to the bedding, including bedding planes as well as jointing. Various vertical to sub-vertical joints, closely to widely spaced, were also noted. No comprehensive stringline fracture data collection was performed for this study. Following presents the orientations of joints, which stood out in the field as comprising the main joint sets (other than the orientation parallel or sub-parallel to the bedding orientation):

J1: N87°W/70°S, N88°E/82°S, N90°E/73°S, N79°W/79°S; N88°E/85°N, N90°E/79°N
J2: N0°W/88°W
J3: N52°E/89°SE
J4: N23°E/48°W
J5: N30°E/73°E
J6: N43°W/68°SW

MICHAEL W. WEST & ASSOCIATES, INC. 6970 S. Holly Circle, Suite 206 Centennial, Colorado 80112-1066	ET-1 Trench Log – Lower Trench 4th Street Bridge – SW Abutment Pueblo, Colorado	BY: PAS, MK	06/28/2006
		PROJ: 05675	FIGURE 2/Sheet 1 of 3



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B1

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B1: About ¾" thick; N30°E/5°E, N26°E/10°E, and N5 °E/7°E
B2: About ⅛" thick
B3: About ¼" to ¾" thick; N2°E/2°E to N25°E/8°E
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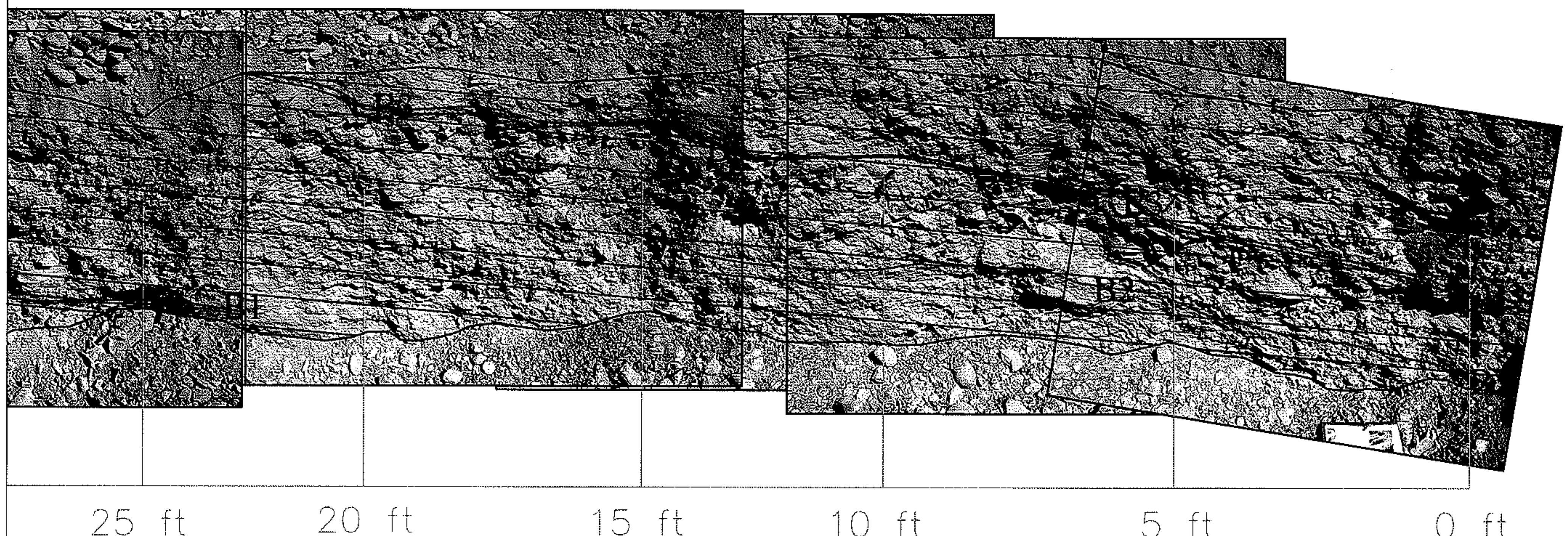
J3: N52°E/89°SE
J4: N23°E/48°W
J5: N30°E/73°E
J6: N43°W/68°SW

MICHAEL W. WEST & ASSOCIATES, INC. 6970 S. Holly Circle, Suite 206 Centennial, Colorado 80112-1066	ET-1 Trench Log - Lower Trench 4th Street Bridge - SW Abutment Pueblo, Colorado	BY: PAS, MK	06/28/2006
		PROJ: 05675	FIGURE 2/Sheet 2 of 3



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B1 ***Bentonite Beds within Shale:*** Orange-brown in color, weathered.

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B2: About ⅛" thick
B3: About ¼" to ¾" thick; N2°E/2°E to N25°E/8°E
B4: About 1 to 1½" thick
B5: About ¾" thick
B6: About 1" thick; N32°E/8°E

Jointing and Fracturing of the Rock: The shale is typically intensely fractured. Very closely to closely spaced fracturing was observed parallel and sub-parallel to the bedding, including bedding planes as well as jointing. Various vertical to sub-vertical joints, closely to widely spaced, were also noted. No comprehensive stringline fracture data collection was performed for this study. Following presents the orientations of joints, which stood out in the field as comprising the main joint sets (other than the orientation parallel or sub-parallel to the bedding orientation):

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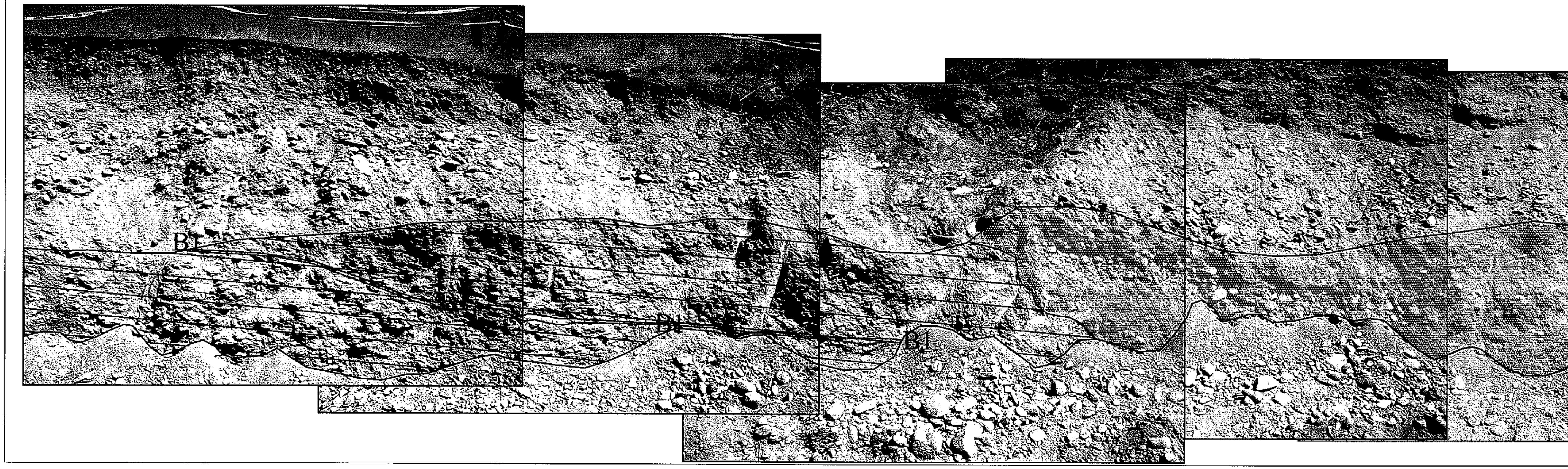
J4: N23°E/48°W
J5: N30°E/73°E
J6: N43°W/68°SW

MICHAEL W. WEST & ASSOCIATES, INC. 6970 S. Holly Circle, Suite 206 Centennial, Colorado 80112-1066	ET–1 Trench Log – Lower Trench 4th Street Bridge – SW Abutment Pueblo, Colorado	BY: PAS, MK	06/28/2006
		PROJ: 05675	FIGURE 2/Sheet 3 of 3



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J3: N52°E/89°SE
J4: N23°E/48°W
J5: N30°E/73°E
J6: N43°W/68°SW

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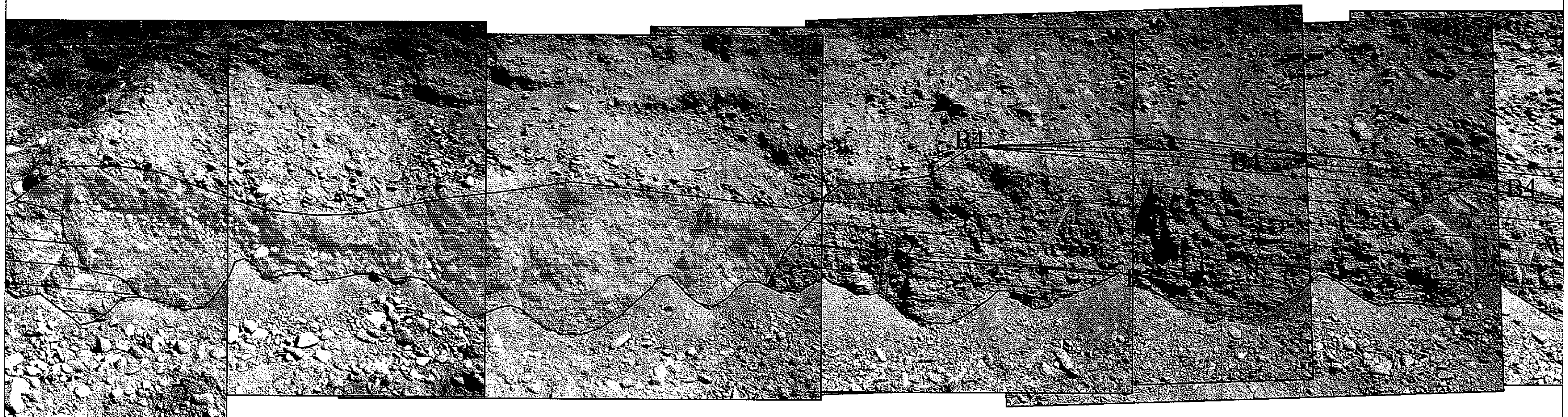
Centennial, Colorado 80112-1066

ET-1 Trench Log – Upper Slope
4th Street Bridge – SW Abutment
Pueblo, Colorado

BY: PAS, MK	06/28/2006
PROJ: 05675	FIGURE 3/Sheet 1 of 3



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B1

Bentonite Beds within Shale: Orange-brown in color, weathered.

B1: About $\frac{3}{4}$" thick; N30°E/5°E, N26°E/10°E, and N5 °E/7°E
B2: About $\frac{1}{8}$" thick
B3: About $\frac{1}{4}$" to $\frac{3}{4}$" thick; N2°E/2°E to N25°E/8°E
B4: About 1 to $1\frac{1}{2}$" thick
B5: About $\frac{3}{4}$" thick
B6: About 1" thick; N32°E/8°E

Jointing and Fracturing of the Rock: The shale is typically intensely fractured. Very closely to closely spaced fracturing was observed parallel and sub-parallel to the bedding, including bedding planes as well as jointing. Various vertical to sub-vertical joints, closely to widely spaced, were also noted. No comprehensive stringline fracture data collection was performed for this study. Following presents the orientations of joints, which stood out in the field as comprising the main joint sets (other than the orientation parallel or sub-parallel to the bedding orientation):

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J3: N52°E/89°SE
J4: N23°E/48°W
J5: N30°E/73°E
J6: N43°W/68°SW

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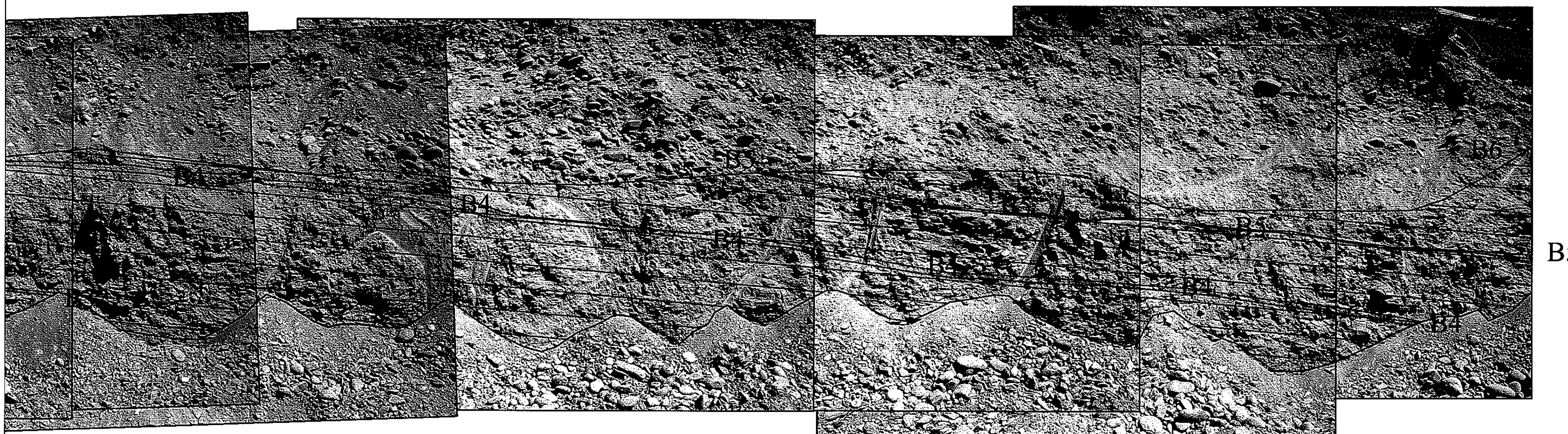
Centennial, Colorado 80112-1066

ET-1 Trench Log – Upper Slope 4th Street Bridge – SW Abutment Pueblo, Colorado	BY: PAS, MK	06/28/2006
	PROJ: 05675	FIGURE 3/Sheet 2 of 3



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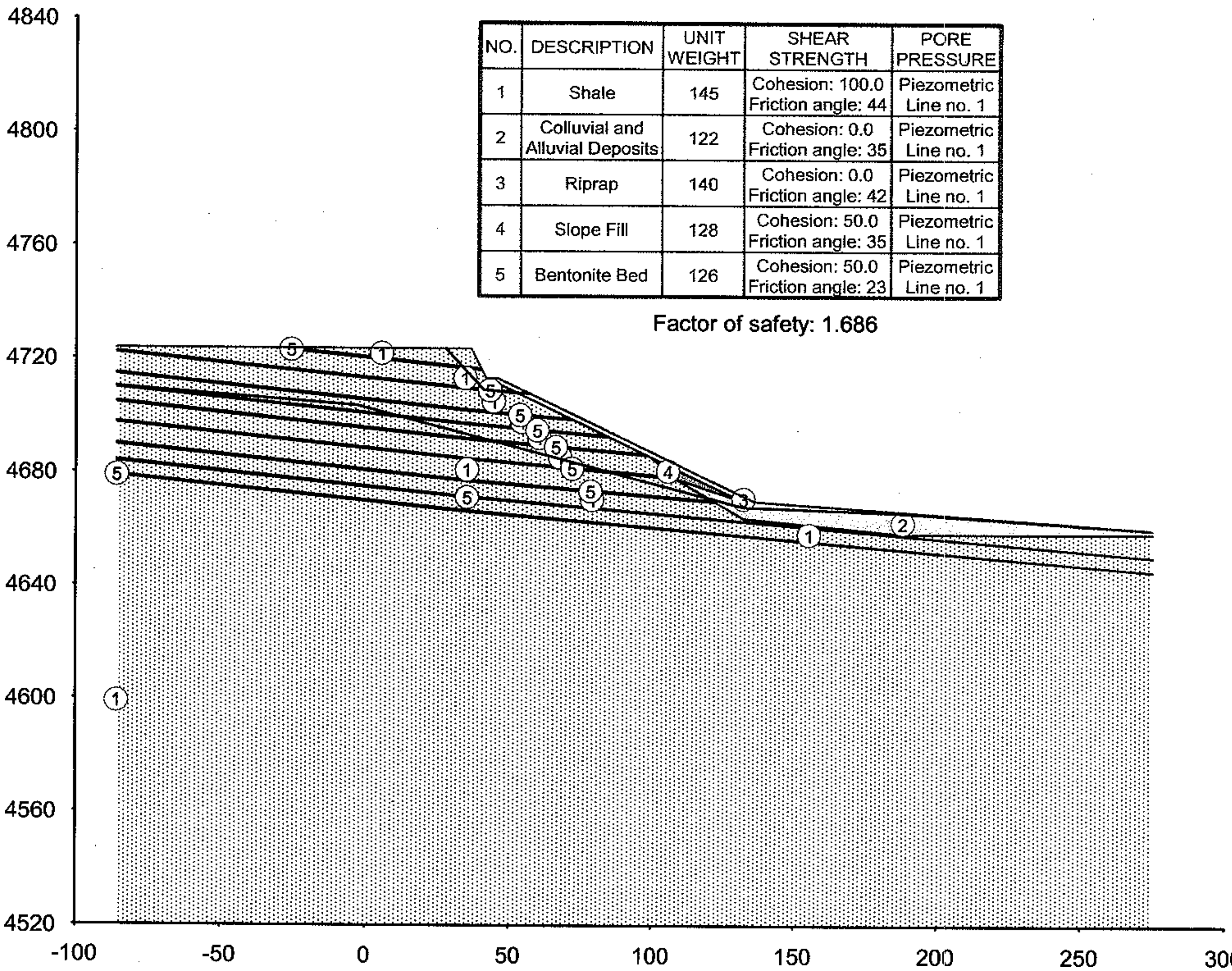
J2: N0°W/88°W
J3: N52°E/89°SE
J4: N23°E/48°W
J5: N30°E/73°E
J6: N43°W/68°SW

MICHAEL W. WEST & ASSOCIATES, INC. 6970 S. Holly Circle, Suite 206 Centennial, Colorado 80112-1066	ET-1 Trench Log – Upper Slope 4th Street Bridge – SW Abutment Pueblo, Colorado	BY: PAS, MK PROJ: 05675	06/28/2006 FIGURE 3/Sheet 3 of 3

4th Street Bridge - SW Abutment - Slope Stability (Case 1 with Bentonite bed)

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

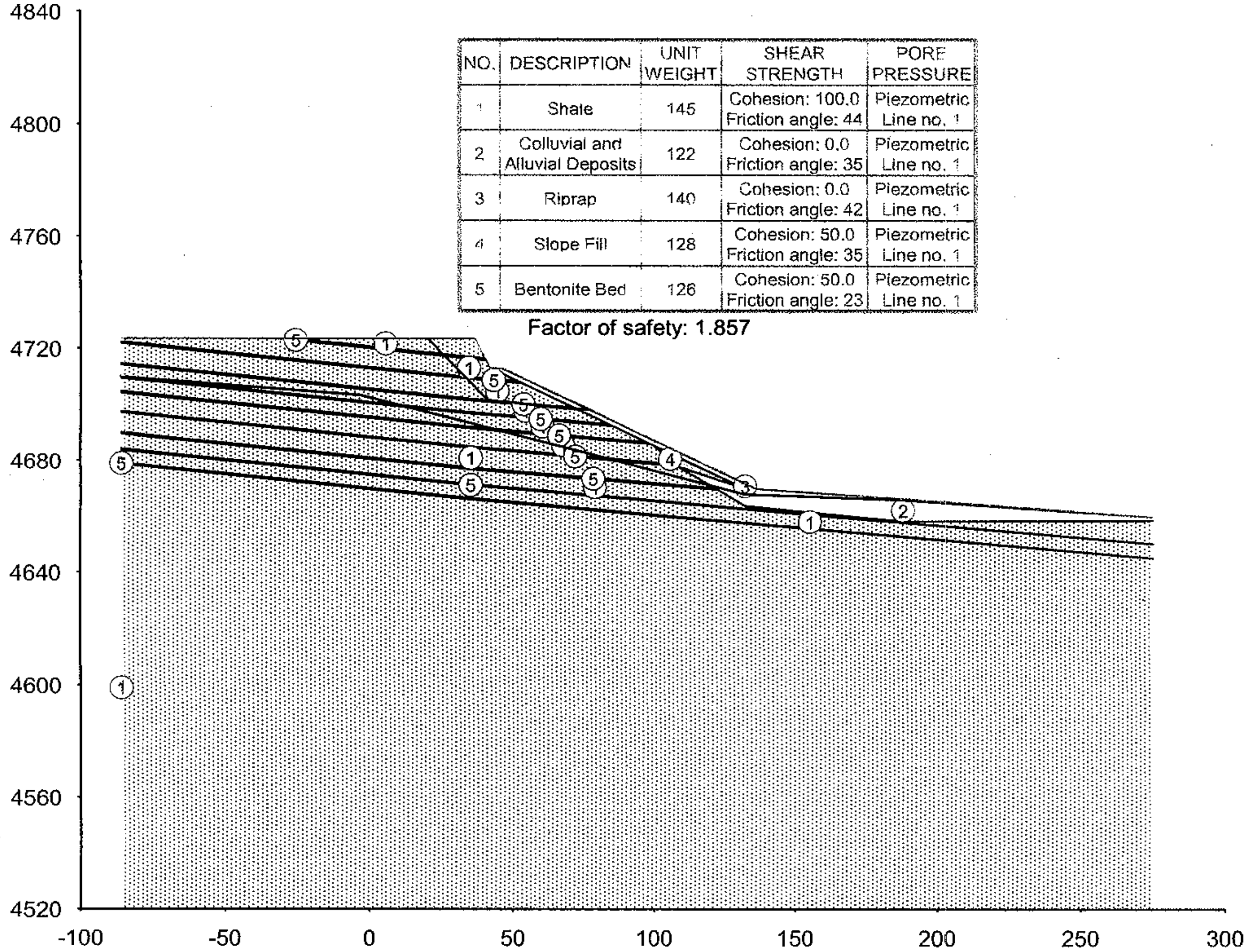
Factor of safety: 1.686



Date: Wed Jan 24 2007 Filename: Time: 09:13:49

Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Noncircular Shear Surface Bentonite Bed Case 1*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 4*

4th Street Bridge - SW Abutment - Slope Stability (Case 2 with Bentonite bed)



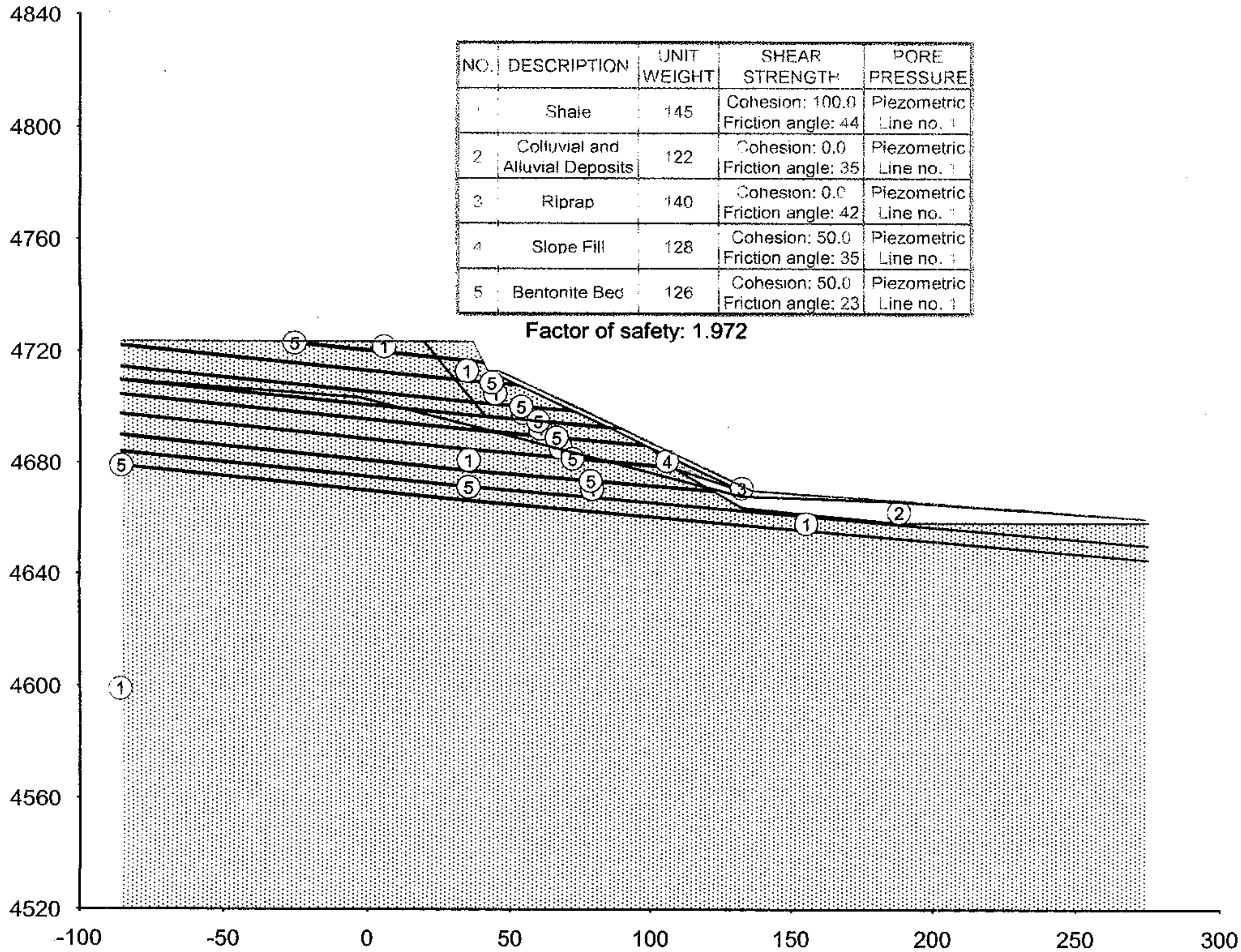
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

Factor of safety: 1.857

Date: Tue Oct 03 2006 Filename: Time: 10:51:46

Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Noncircular Shear Surface Bentonite Bed Case 2*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 5*

4th Street Bridge - SW Abutment - Slope Stability (Case 3 with Bentonite bed)

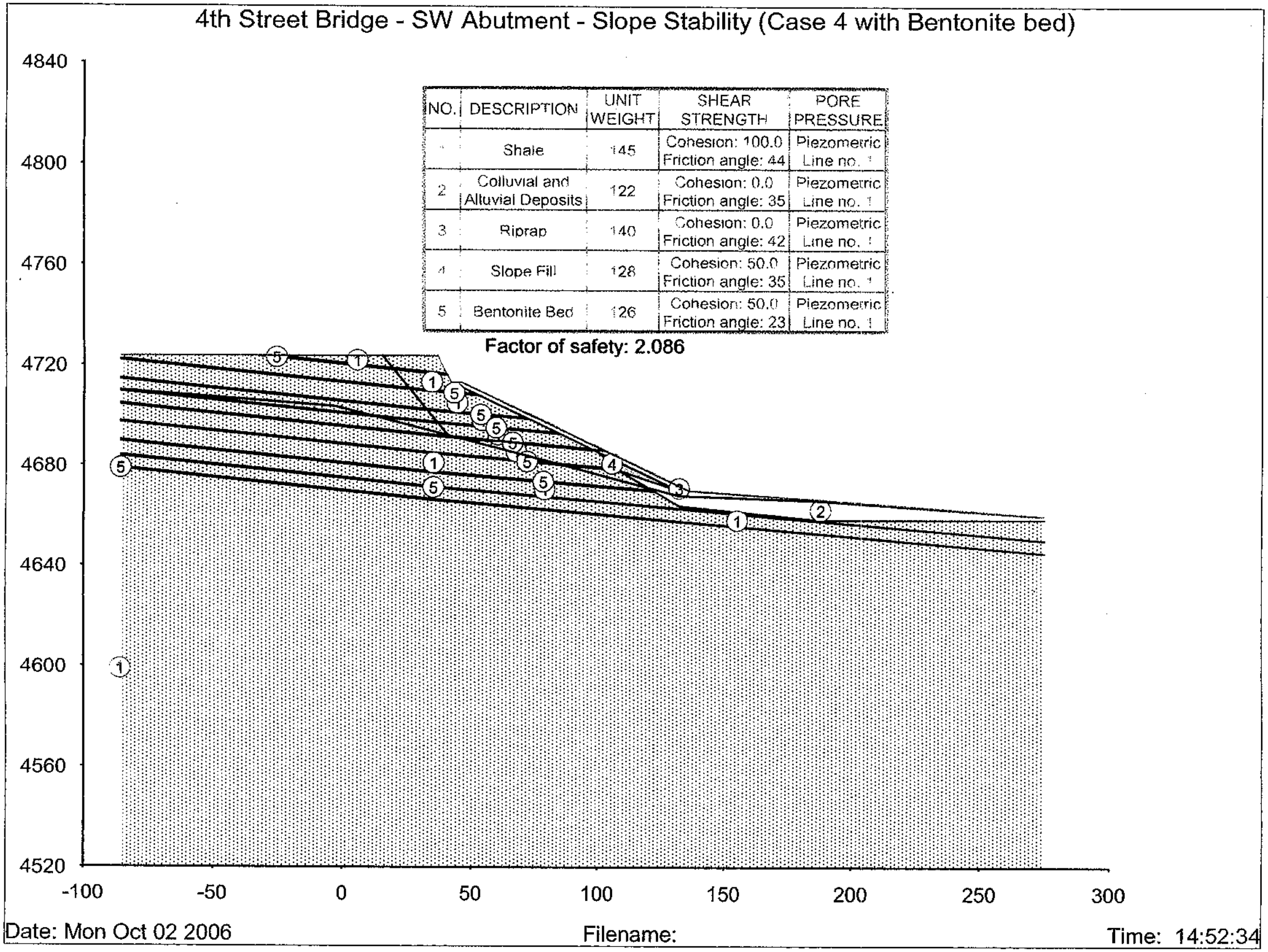


NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

Factor of safety: 1.972

Date: Mon Oct 02 2006 Filename: Time: 14:44:49

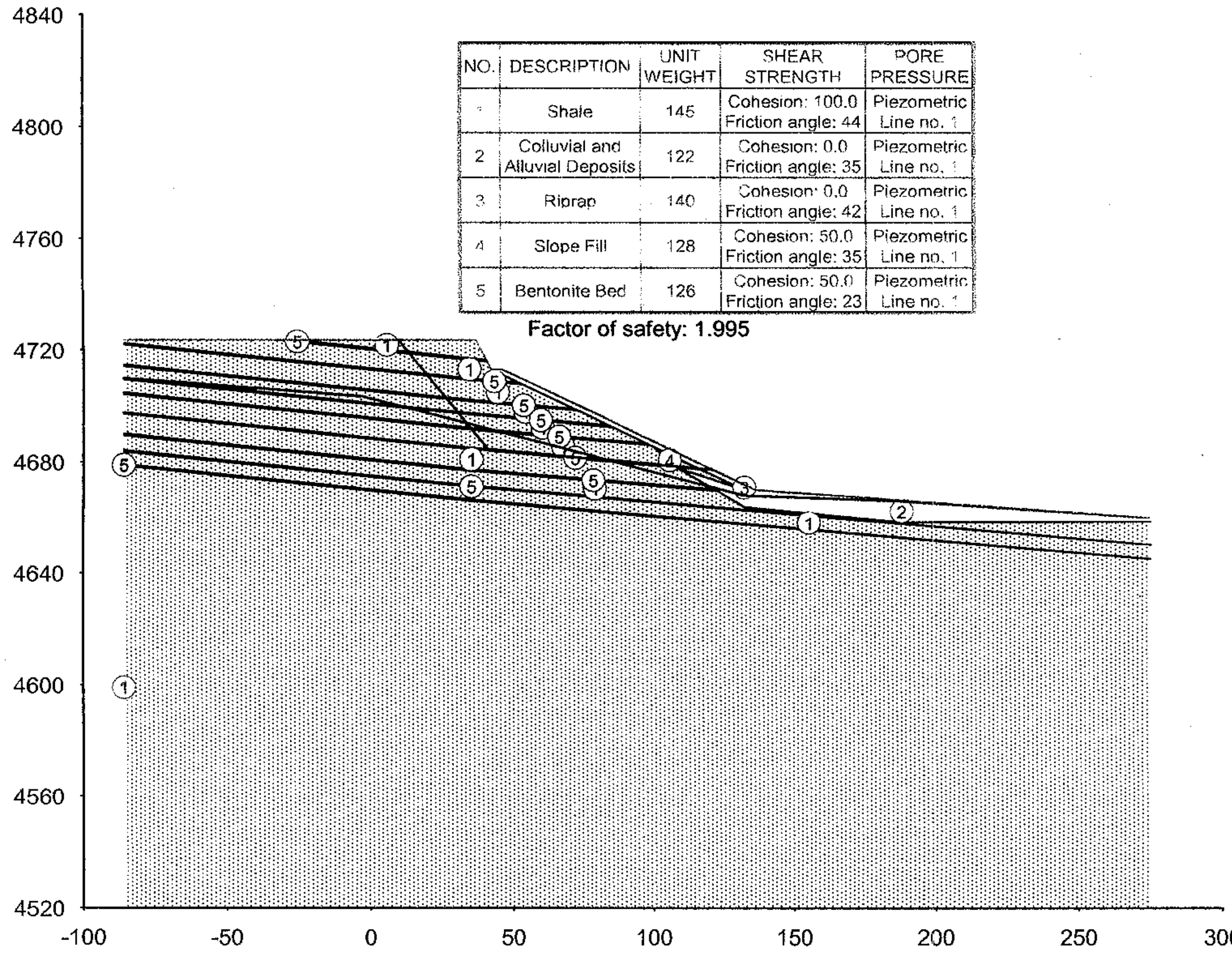
Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Noncircular Shear Surface Bentonite Bed Case 3*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 6*



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Noncircular Shear Surface Bentonite Bed Case 4*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 7*

4th Street Bridge - SW Abutment - Slope Stability (Case 5 with Bentonite bed)



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

Factor of safety: 1.995

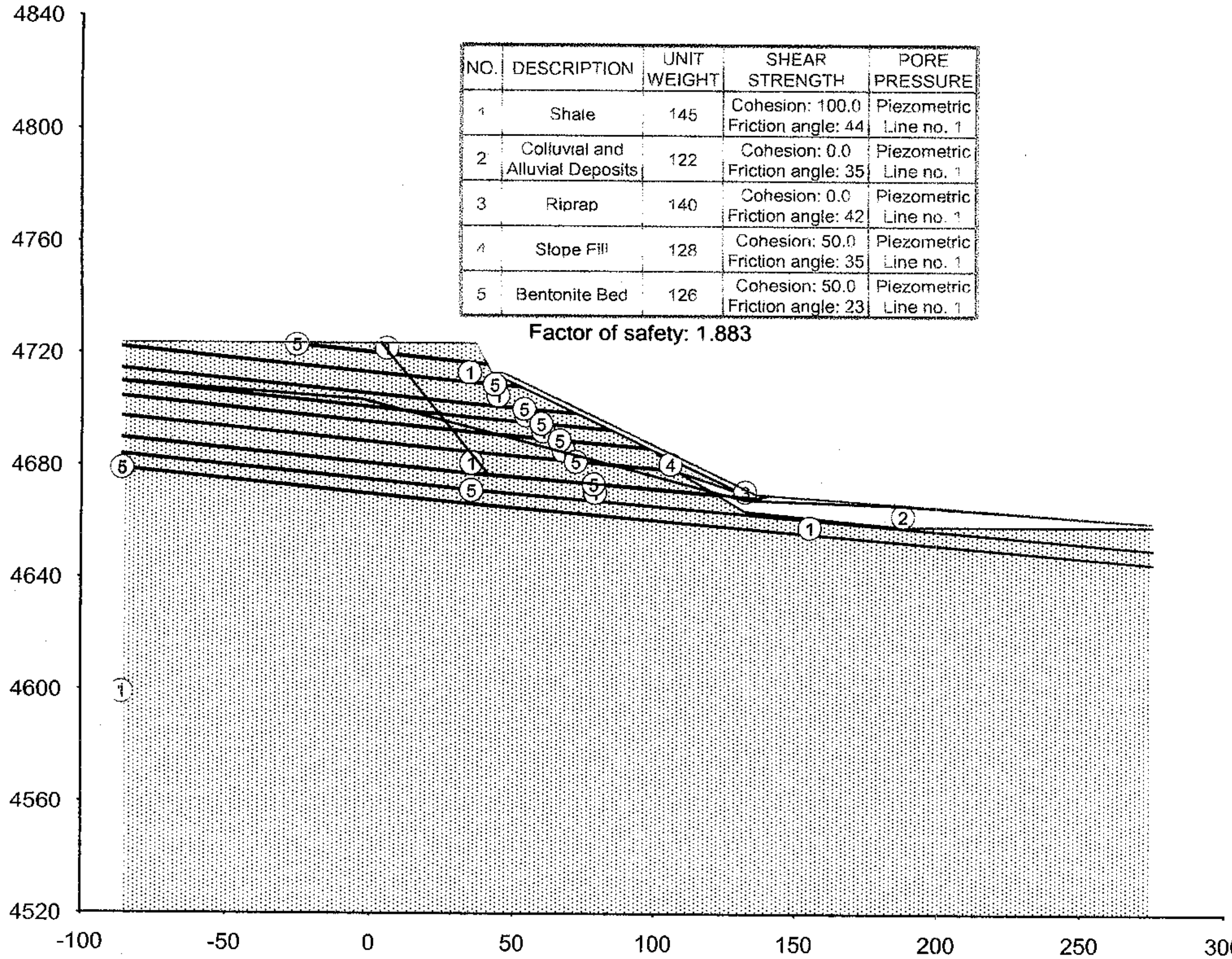
Date: Tue Oct 10 2006 Filename: Time: 09:08:14

Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Noncircular Shear Surface Bentonite Bed Case 5*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 8*

4th Street Bridge - SW Abutment - Slope Stability (Case 6 with Bentonite bed)

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

Factor of safety: 1.883



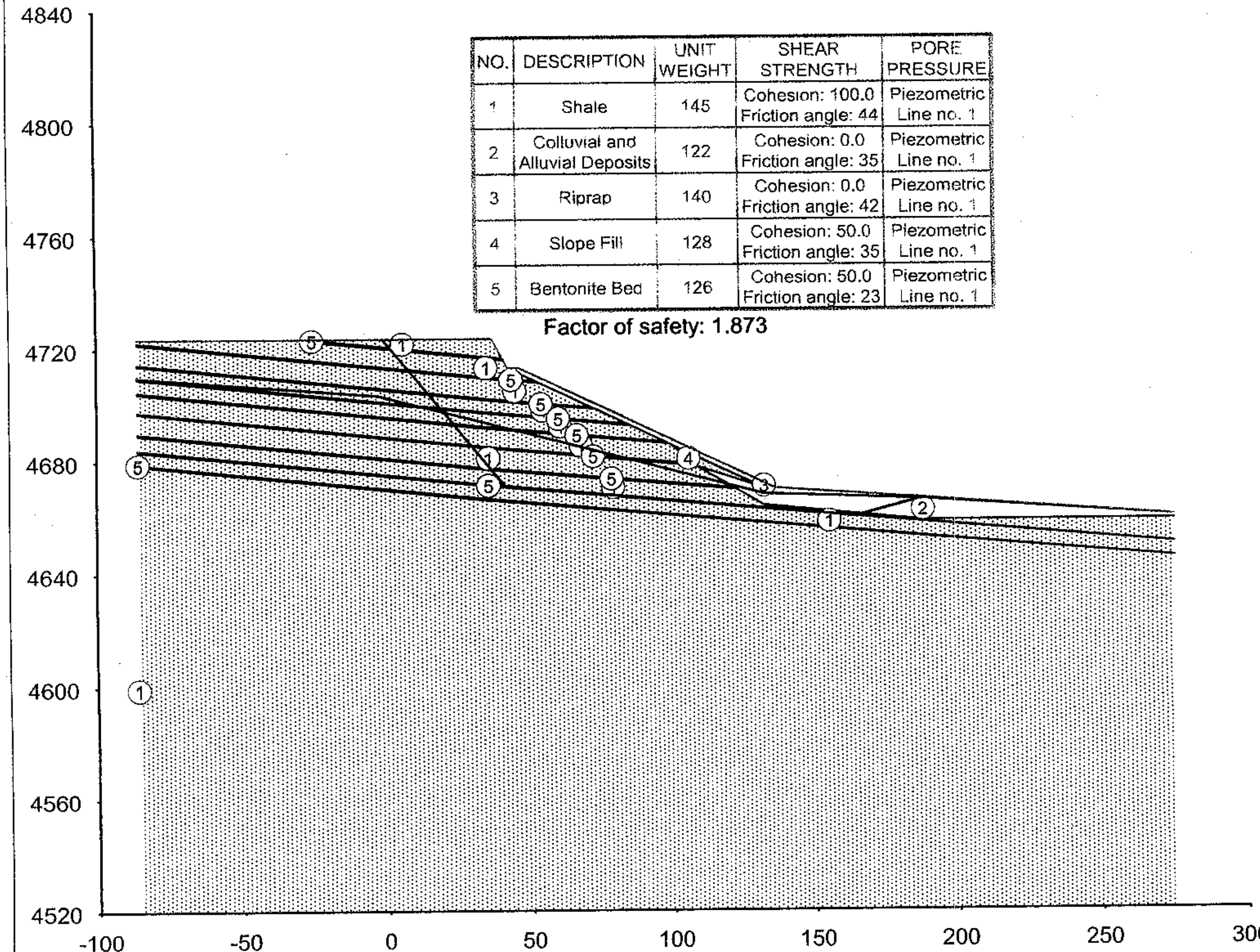
Date: Mon Oct 02 2006 Filename: Time: 15:25:00

Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Noncircular Shear Surface Bentonite Bed Case 6*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 9*

4th Street Bridge - SW Abutment - Slope Stability (Case 7 with Bentonite bed)

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

Factor of safety: 1.873



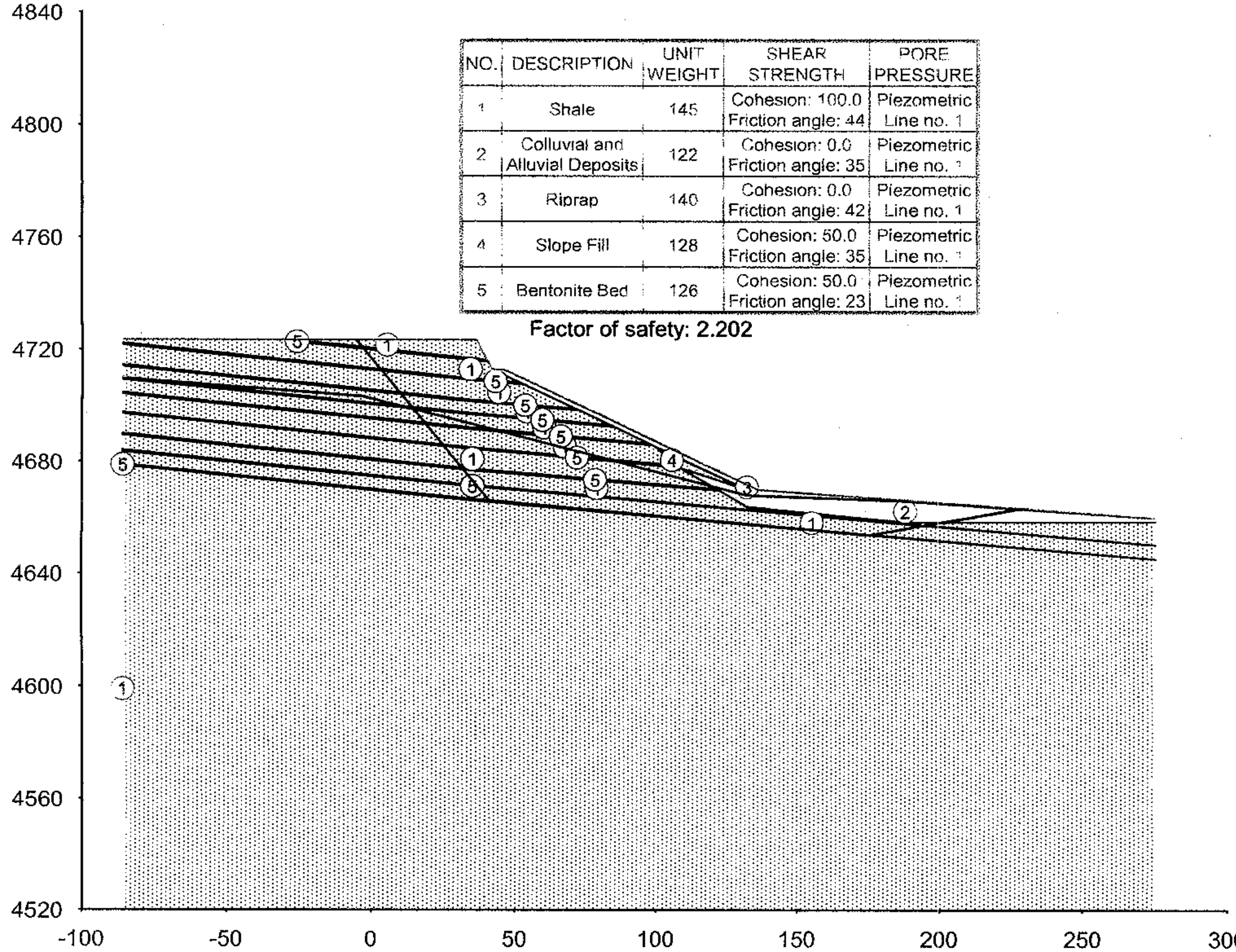
Date: Tue Oct 10 2006 Filename: Time: 09:48:27

Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Noncircular Shear Surface Bentonite Bed Case 7*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 10*

4th Street Bridge - SW Abutment - Slope Stability (Case 8 with Bentonite bed)

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

Factor of safety: 2.202



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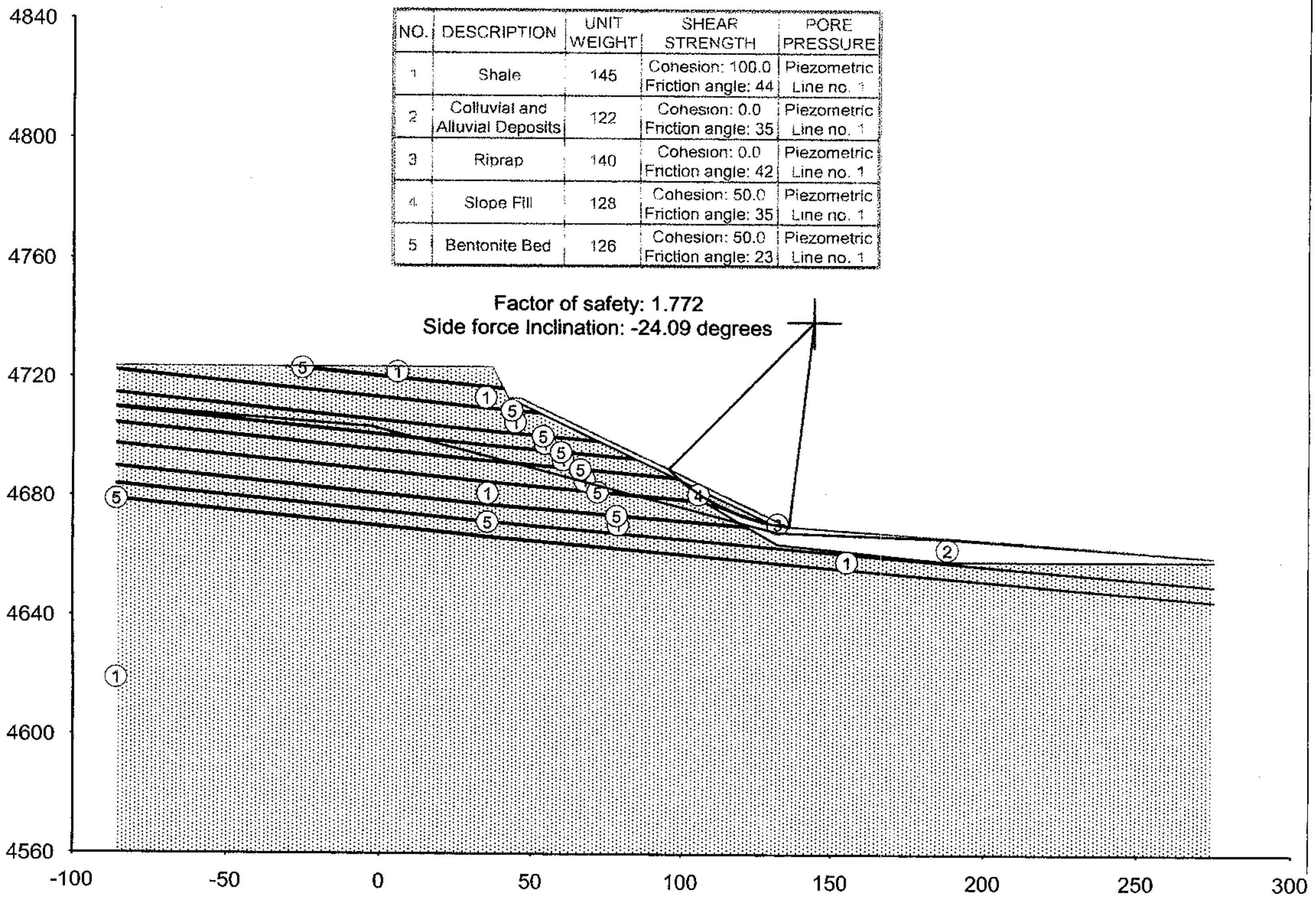
Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Noncircular Shear Surface Bentonite Bed Case 8*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 11*

4th Street Bridge - SW Abutment - Slope Stability (Case 5 with Bentonite bed)

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1

Factor of safety: 1.772
Side force Inclination: -24.09 degrees



Date: Tue Oct 03 2006

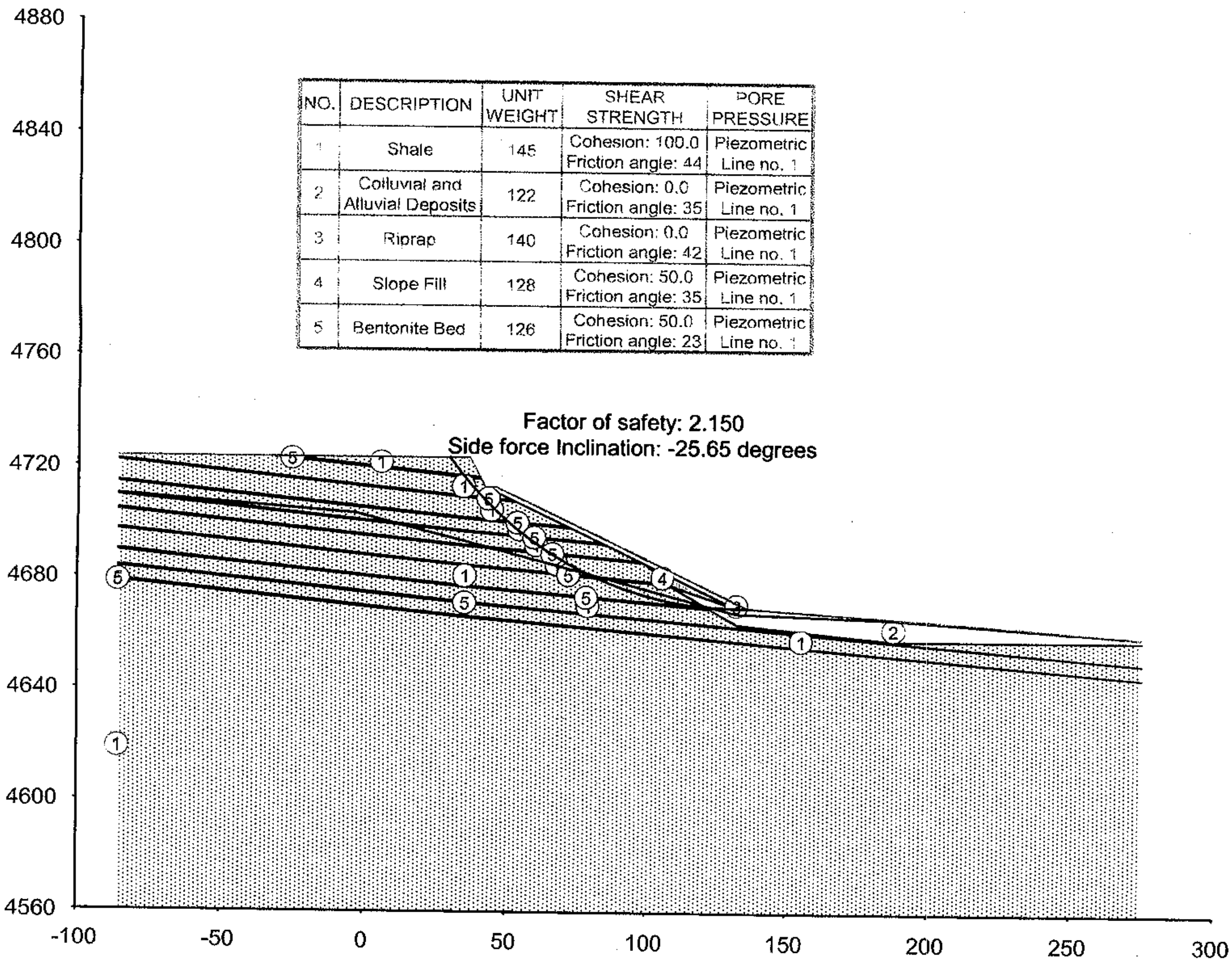
Filename:

Time: 11:26:26

Michael W. West and Associates, Inc. *Consulting Engineers and Geologists*	*Slope Stability Analysis Circular Shear Surface Case 1*	*BY: VB*	*01/24/2007*
		Proj.: 05675	*FIGURE 12*

4th Street Bridge - SW Abutment - Slope Stability (Case 5 with Bentonite bed)

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Shale	145	Cohesion: 100.0 Friction angle: 44	Piezometric Line no. 1
2	Colluvial and Alluvial Deposits	122	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
3	Riprap	140	Cohesion: 0.0 Friction angle: 42	Piezometric Line no. 1
4	Slope Fill	128	Cohesion: 50.0 Friction angle: 35	Piezometric Line no. 1
5	Bentonite Bed	126	Cohesion: 50.0 Friction angle: 23	Piezometric Line no. 1



Date: Tue Oct 03 2006

Filename:

Time: 11:27:11

Michael W. West and Associates, Inc.
Consulting Engineers and Geologists

Slope Stability Analysis
Circular Shear Surface
Case 2

BY: VB

01/24/2007

Proj.: 05675

FIGURE 13

ATTACHMENT A - LABORATORY TEST RESULTS

ATTERBERG LIMITS
ASTM D 4318

ATTERBERG LIMITS TEST
ASTM D 4318

CLIENT	Goodson & Associates	JOB NO.	2014-104
BORING NO.	BR-1	DATE SAMPLED	
DEPTH	42.0-42.2	DATE TESTED	06-21-06 WAR
SAMPLE NO.			
SOIL DESCR.			
LOCATION	4th St. Bridge, SH 96		

Plastic Limit
Determination

Wt Dish & Wet Soil
Wt Dish & Dry Soil
Wt of Moisture
Wt of Dish
Wt of Dry Soil
Moisture Content

NON-PLASTIC

Liquid Limit
Determination

Device Number 0866

Number of Blows

Wt Dish & Wet Soil
Wt Dish & Dry Soil
Wt of Moisture
Wt of Dish
Wt of Dry Soil
Moisture Content

NON-PLASTIC

Liquid Limit	NP
Plastic Limit	NP
Plasticity Index	NP

Atterberg Classification NP

Data entry by: SR Date: 06/21/2006
Checked by: RS Date: 6/22/06
FileName: GDG04204

ADVANCED TERRA TESTING, INC.

ATTERBERG LIMITS TEST
ASTM D 4318

CLIENT	Goodson & Associates	JOB NO.	2014-104
BORING NO.	BR-2	DATE SAMPLED	
DEPTH	75.5-76.2	DATE TESTED	07-12-06 SR
SAMPLE NO.	2" Bentonite Seam		
SOIL DESCR.			
LOCATION	4th Street Bridge SH 96		

Plastic Limit Determination

	1	2	3
Wt Dish & Wet Soil	5.46	6.81	6.80
Wt Dish & Dry Soil	4.84	5.99	6.00
Wt of Moisture	0.62	0.82	0.80
Wt of Dish	0.82	0.76	0.75
Wt of Dry Soil	4.02	5.23	5.25
Moisture Content	15.42	15.68	15.24

Liquid Limit Determination   Device Number 0966

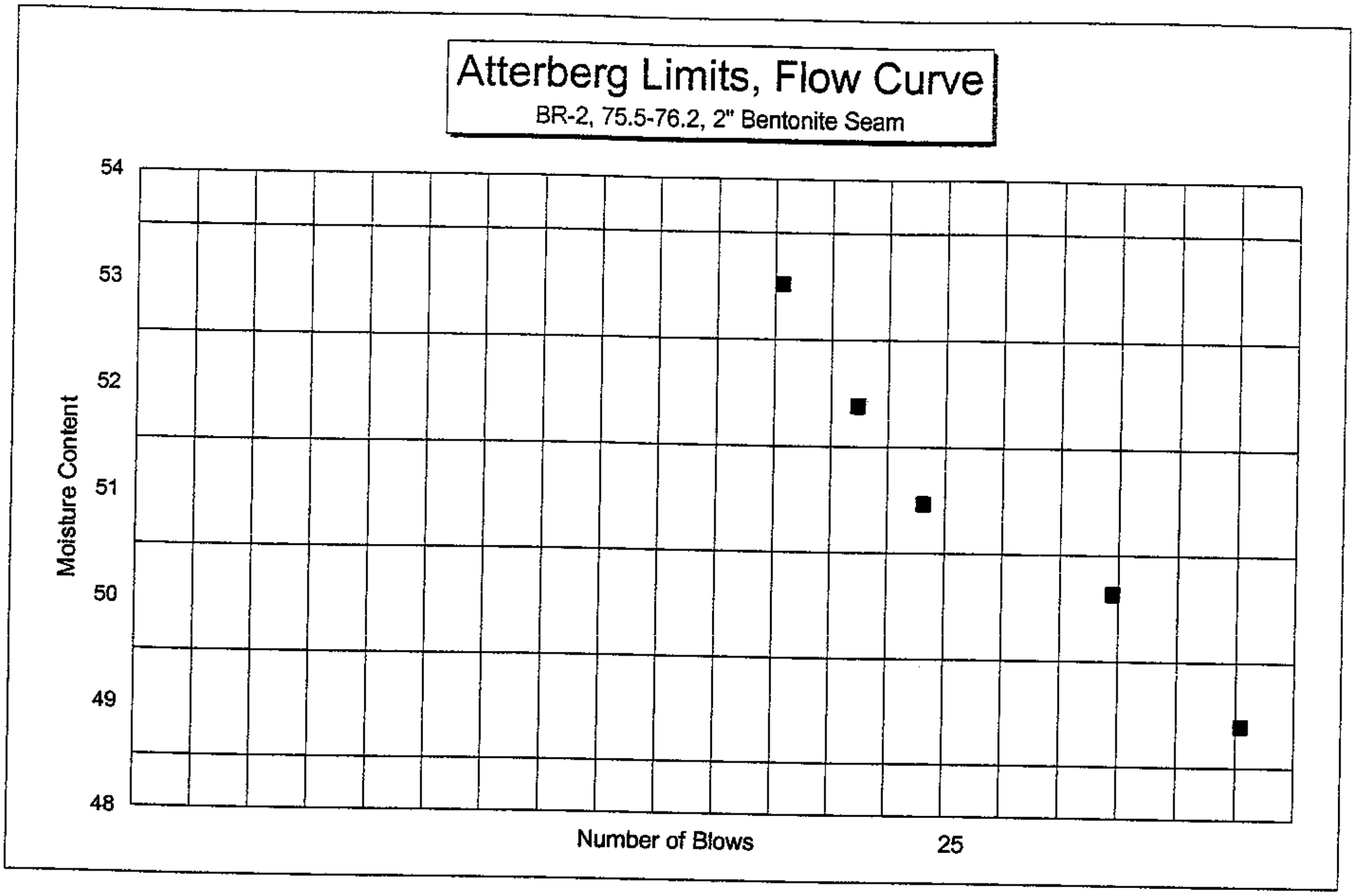
	1	2	3	4	5
Number of Blows	45	35	24	21	18
Wt Dish & Wet Soil	11.53	12.10	12.49	11.63	13.42
Wt Dish & Dry Soil	7.99	8.31	8.53	7.91	9.03
Wt of Moisture	3.54	3.79	3.96	3.72	4.39
Wt of Dish	0.75	0.75	0.76	0.74	0.75
Wt of Dry Soil	7.24	7.56	7.77	7.17	8.28
Moisture Content	48.90	50.13	50.97	51.88	53.02

Liquid Limit	51.3
Plastic Limit	15.4
Plasticity Index	35.8

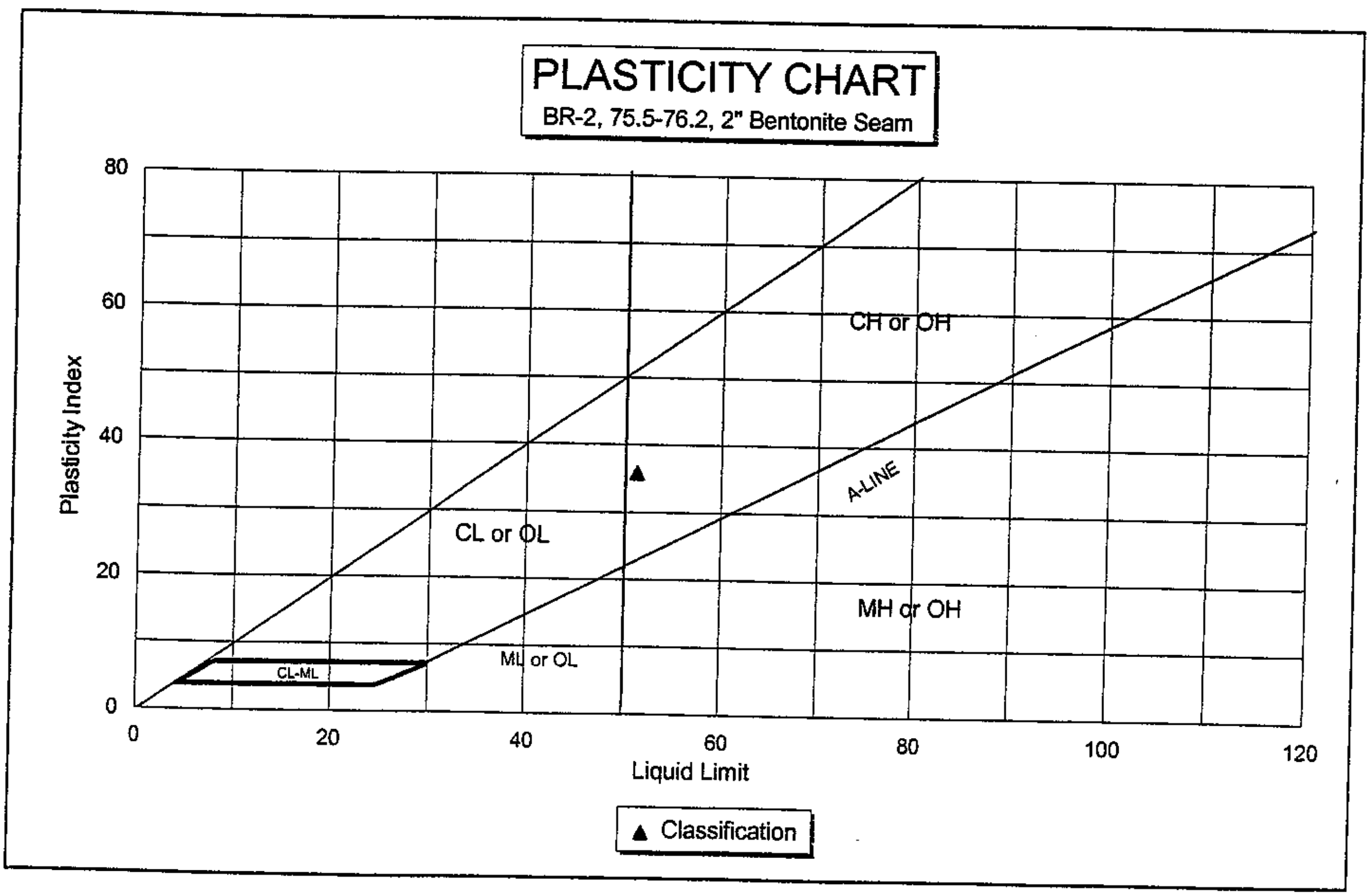
Atterberg Classification CH

Data entry by: SR Date: 07/13/2006
Checked by: RS Date: 7/13/06
FileName: GDG07557

ADVANCED TERRA TESTING, INC.



Atterberg Limits, Flow Curve
BR-2, 75.5-76.2, 2" Bentonite Seam
Moisture Content
54
53
52
51
50
49
48
Number of Blows
25



PLASTICITY CHART
BR-2, 75.5-76.2, 2" Bentonite Seam
Plasticity Index
80
60
40
20
0
CH or OH
A-LINE
CL or OL
MH or OH
ML or OL
CL-ML
0
20
40
60
80
100
120
Liquid Limit
▲ Classification

ATTERBERG LIMITS TEST
ASTM D 4318

CLIENT	Goodson & Associates	JOB NO.	2014-104
BORING NO.	BR-1	DATE SAMPLED	
DEPTH	15	DATE TESTED	07-13-06 WAR
SAMPLE NO.			
SOIL DESCR.			
LOCATION	4th Street Bridge SH 96		

Plastic Limit Determination

	1	2	3
Wt Dish & Wet Soil	7.50	8.64	7.67
Wt Dish & Dry Soil	6.25	7.16	6.35
Wt of Moisture	1.25	1.48	1.32
Wt of Dish	0.76	0.81	0.75
Wt of Dry Soil	5.49	6.35	5.60
Moisture Content	22.77	23.31	23.57

Liquid Limit Determination   Device Number   0860

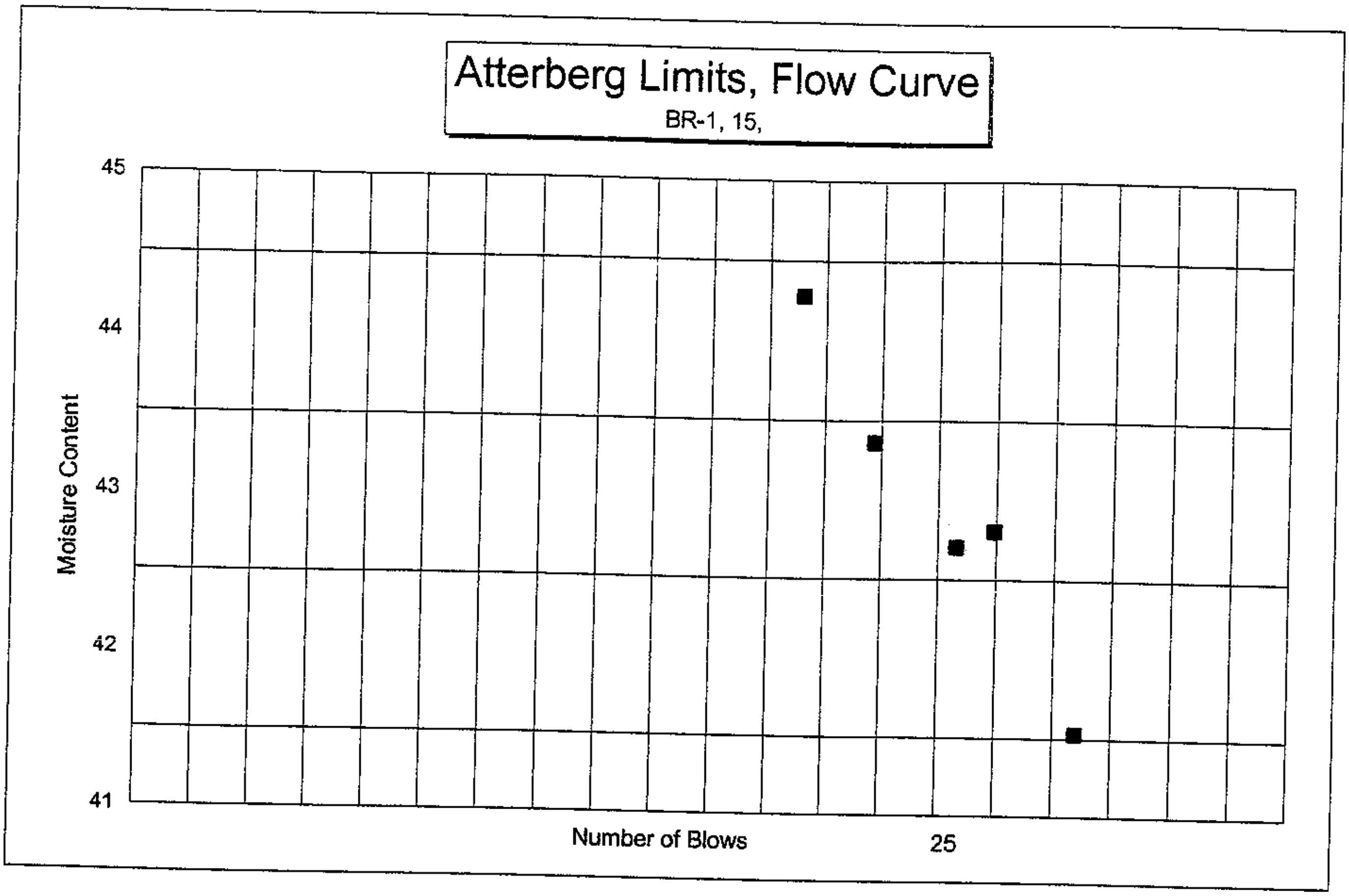
	1	2	3	4	5
Number of Blows	33	28	19	22	26
Wt Dish & Wet Soil	7.84	9.09	8.45	9.27	9.04
Wt Dish & Dry Soil	5.78	6.59	6.09	6.69	6.58
Wt of Moisture	2.06	2.50	2.36	2.58	2.46
Wt of Dish	0.82	0.75	0.76	0.74	0.82
Wt of Dry Soil	4.96	5.84	5.33	5.95	5.76
Moisture Content	41.53	42.81	44.28	43.36	42.71

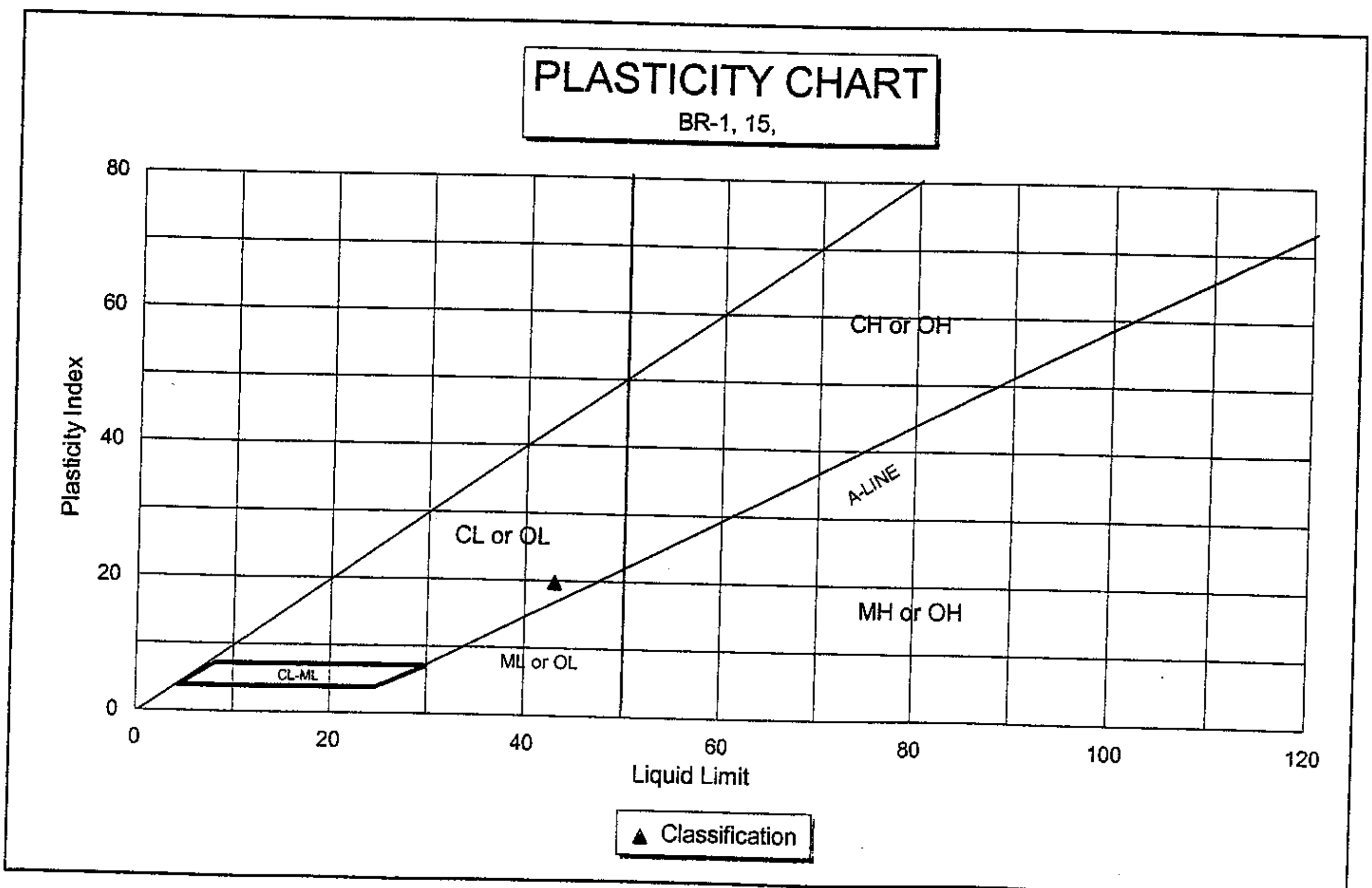
Liquid Limit	43.0
Plastic Limit	23.2
Plasticity Index	19.7

Atterberg Classification   CL

Data entry by: SR   Date: 07/14/2006
Checked by: RB   Date: 7/14/06
FileName: GDG0BR1

ADVANCED TERRA TESTING, INC.


Atterberg Limits, Flow Curve
BR-1, 15,
Moisture Content
45
44
43
42
41
Number of Blows
25


PLASTICITY CHART
BR-1, 15,
Plasticity Index
80
60
40
20
0
CH or OH
A-LINE
CL or OL
MH or OH
ML or OL
CL-ML
0
20
40
60
80
100
120
Liquid Limit
Classification

ATTERBERG LIMITS TEST
ASTM D 4318

CLIENT	Goodson & Assocites	JOB NO.	2014-104
BORING NO.	BR-2	DATE SAMPLED	
DEPTH	38.5-39.5'	DATE TESTED	07-07-06 WAR
SAMPLE NO.	Rock Core		
SOIL DESCR.			
LOCATION	4th Street Bridge SH 96		

Plastic Limit Determination

	1	2	3
Wt Dish & Wet Soil	8.74	7.20	8.17
Wt Dish & Dry Soil	7.69	6.40	7.26
Wt of Moisture	1.05	0.80	0.91
Wt of Dish	0.81	0.82	0.82
Wt of Dry Soil	6.88	5.58	6.44
Moisture Content	15.26	14.34	14.13

Liquid Limit Determination   Device Number 0860

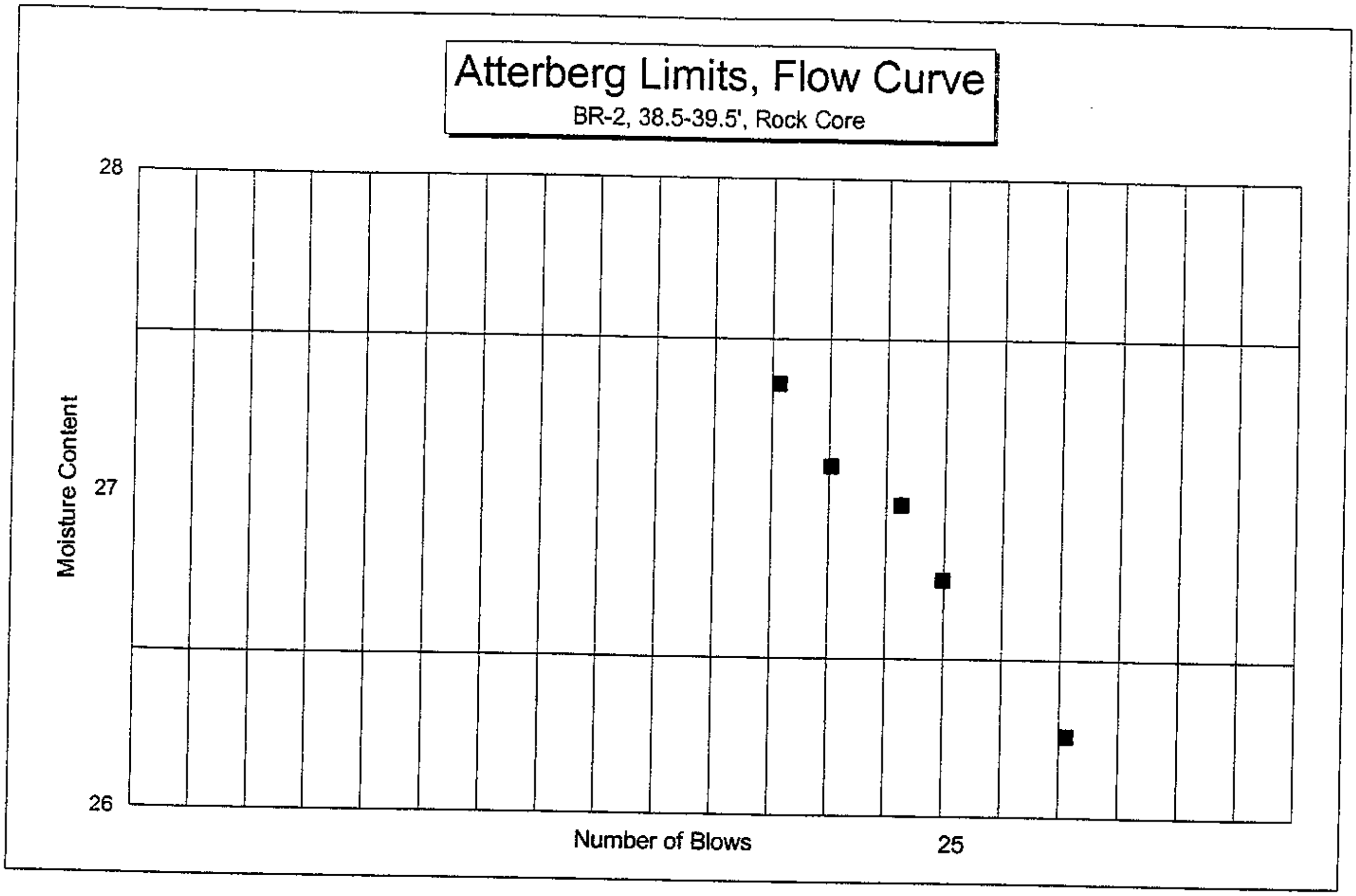
	1	2	3	4	5
Number of Blows	18	20	25	32	23
Wt Dish & Wet Soil	8.78	9.73	10.06	11.60	9.61
Wt Dish & Dry Soil	7.07	7.83	8.11	9.36	7.74
Wt of Moisture	1.71	1.90	1.95	2.24	1.87
Wt of Dish	0.82	0.82	0.82	0.83	0.81
Wt of Dry Soil	6.25	7.01	7.29	8.53	6.93
Moisture Content	27.36	27.10	26.75	26.26	26.98

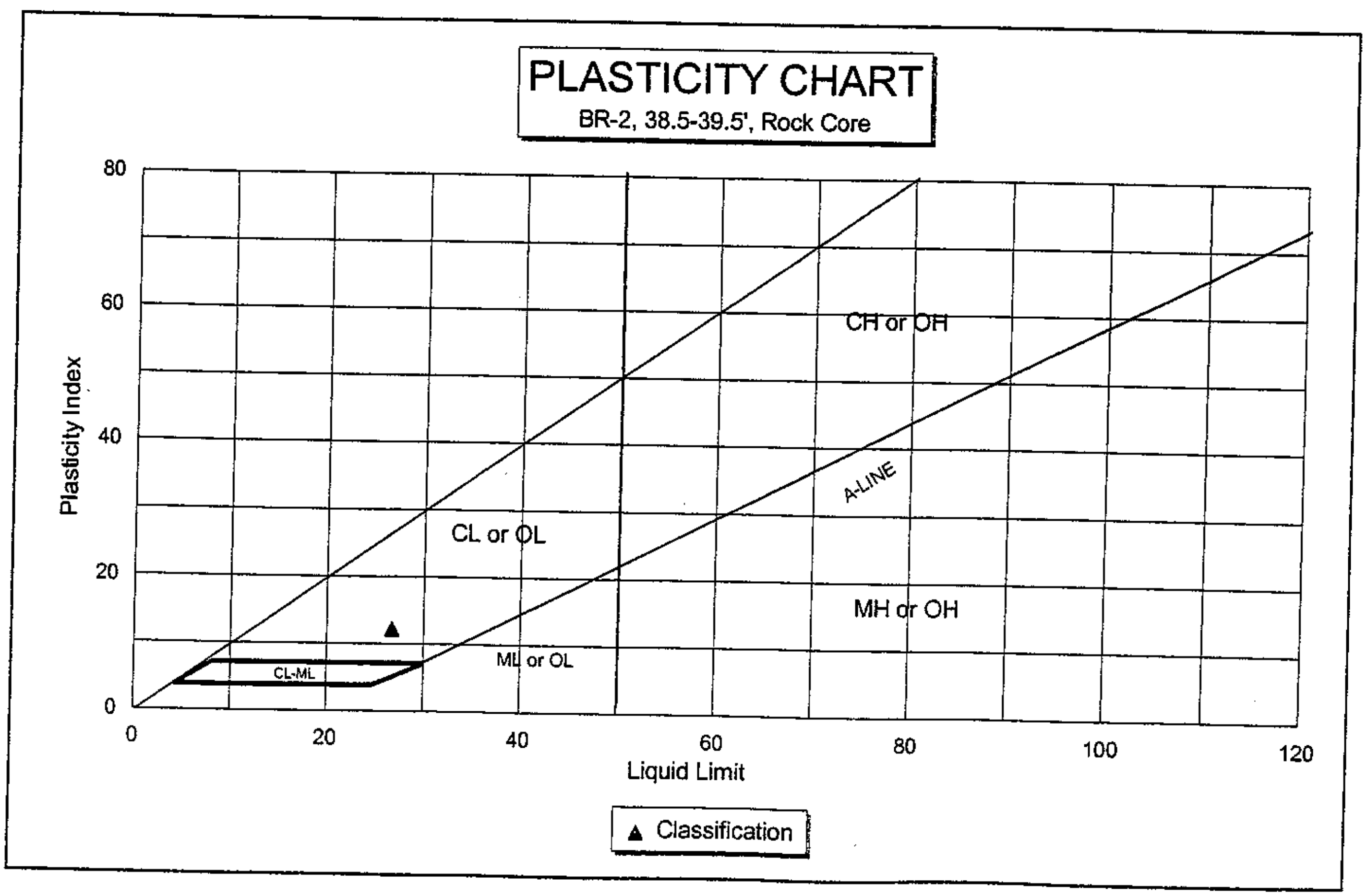
Liquid Limit	26.7
Plastic Limit	14.6
Plasticity Index	12.2

Atterberg Classification CL

Data entry by: SR Date: 07/10/2006
Checked by: RD Date: 7/10/06
FileName: GDG03853

ADVANCED TERRA TESTING, INC.


Atterberg Limits, Flow Curve
BR-2, 38.5-39.5', Rock Core
28
27
26
Moisture Content
Number of Blows
25


PLASTICITY CHART
BR-2, 38.5-39.5', Rock Core
80
60
40
20
0
Plasticity Index
0
20
40
60
80
100
120
Liquid Limit
CH or OH
A-LINE
CL or OL
MH or OH
ML or OL
CL-ML
▲ Classification

GRAIN SIZE ANALYSIS
3 Inch To - #200
ASTM D 422

MECHANICAL ANALYSIS - SIEVE TEST DATA
ASTM D-1140

CLIENT Goodson & Associates

JOB NO. 2014-104

BORING NO.	BR-2
DEPTH	38.5-39.5'
SAMPLE NO.	Rock Core
SOIL DESCR.	
LOCATION	4th Street Bridge SH 96

SAMPLED	
DATE TESTED	07-06-06 CAL
WASH SIEVE	Yes
DRY SIEVE	No

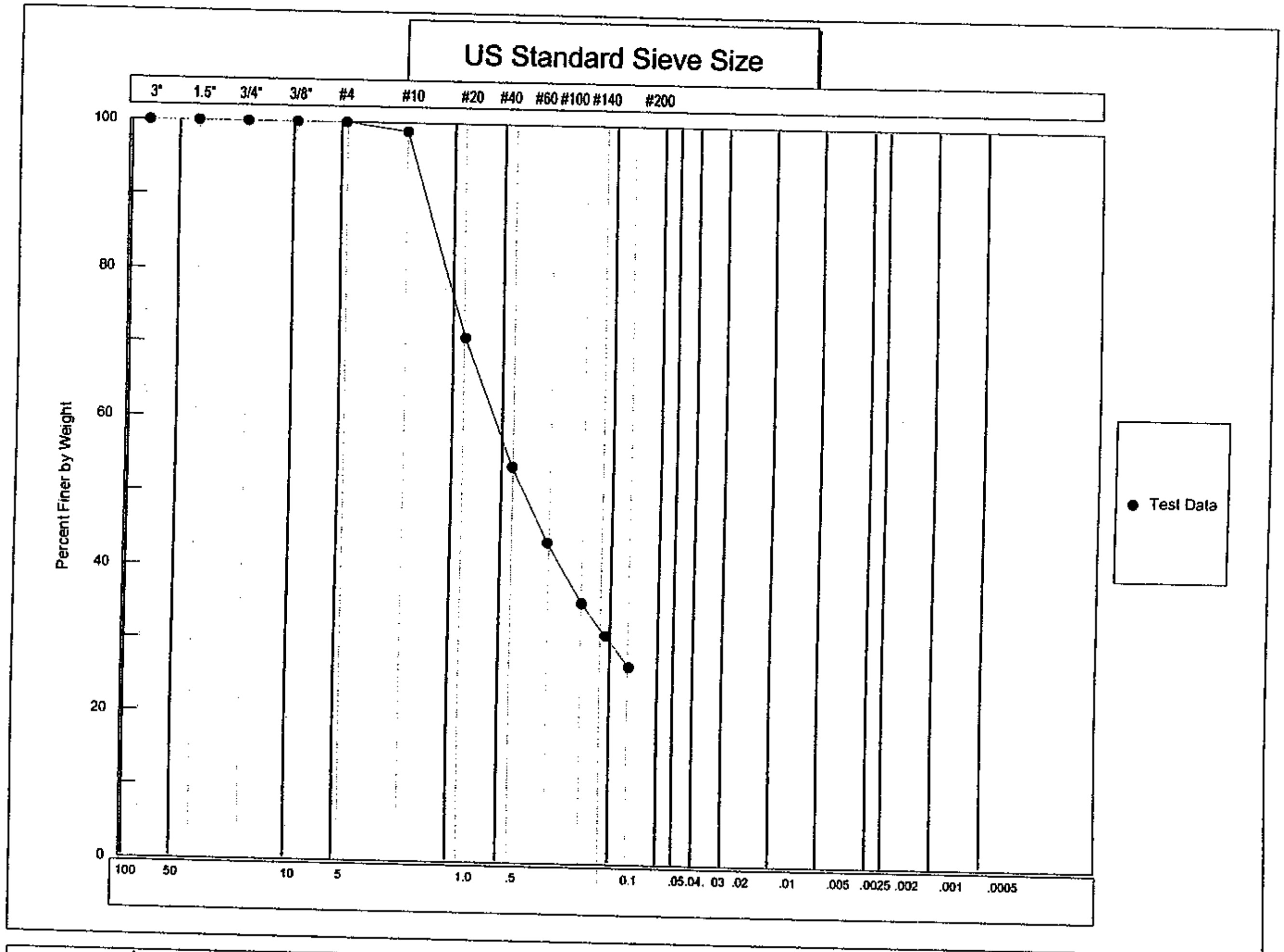
WASH SIEVE ANALYSIS

Wt. Wet Soil & Pan Before Washing (g)	118.6
Wt. Dry Soil & Pan Before Washing (g)	116.9
Weight of Pan (g)	8.2
Wt. of Dry Soil Before Washing	108.8
Wt. Dry Soil & Pan After Washing (g)	83.1
Wt. of Dry Soil After Washing (g)	74.9
-#200 Wash. Out %	31.1

Sieve Number (Size)	Pan Weight (g)	Indiv. Wt. + Pan (g)	Indiv. Wt. Retain.	Cum. Wt. Retain.	Cum. % Retain.	% Finer By Wt.
3"	0.00	0.00	0.00	0.00	0.0	100.0
1 1/2"	0.00	0.00	0.00	0.00	0.0	100.0
3/4"	0.00	0.00	0.00	0.00	0.0	100.0
3/8"	0.00	0.00	0.00	0.00	0.0	100.0
#4	0.00	0.00	0.00	0.00	0.0	100.0
#10	3.63	4.96	1.33	1.33	1.2	98.8
#20	3.69	33.92	30.23	31.56	29.0	71.0
#40	3.68	22.42	18.74	50.30	46.2	53.8
#60	3.80	14.80	11.00	61.30	56.4	43.6
#100	3.73	12.69	8.96	70.26	64.6	35.4
#140	3.70	8.35	4.65	74.91	68.9	31.1
#200	3.75	8.37	4.62	79.53	73.1	26.9

Data entered by: SR Date: 07/07/2006
Data checked by: RS Date: 7/10/06
FileName: GOM0BR2

ADVANCED TERRA TESTING, INC.



COBBLES	GRAVEL		SAND			SILT OR CLAY	USCS
	COARSE	FINE	CRS	MEDIUM	FINE		

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	WENTWORTH
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			

Client: Goodson & Associates Boring No.: BR-2
Job Number: 2014-104
Classification______________

Depth: 38.5-39.5'

Sample No.: Rock Core

Advanced Terra Testing, Inc.

MECHANICAL ANALYSIS - SIEVE TEST DATA
ASTM D 6913

CLIENT Goodson & Associates

JOB NO. 2014-104

BORING NO. BR-1
DEPTH 15'
SAMPLE NO.
SOIL DESCR.
LOCATION 4th Street Bridge SH 96

SAMPLED
DATE+#4 WASHED 07-11-06 WAR
DATE -#4 WASHED 07-11-06 WAR
WASH SIEVE Yes
DRY SIEVE No

MOISTURE DATA

HYGROSCOPIC Yes

NATURAL No

Wt. Wet Soil & Pan (g)	47.29
Wt. Dry Soil & Pan (g)	45.67
Wt. Lost Moisture (g)	1.62
Wt. of Pan Only (g)	3.73
Wt. of Dry Soil (g)	41.94
Moisture Content %	3.9
Wt. Partial -#4 Sample Wet (g)	119.67
Wt. Partial Sample Dry (g)	115.22

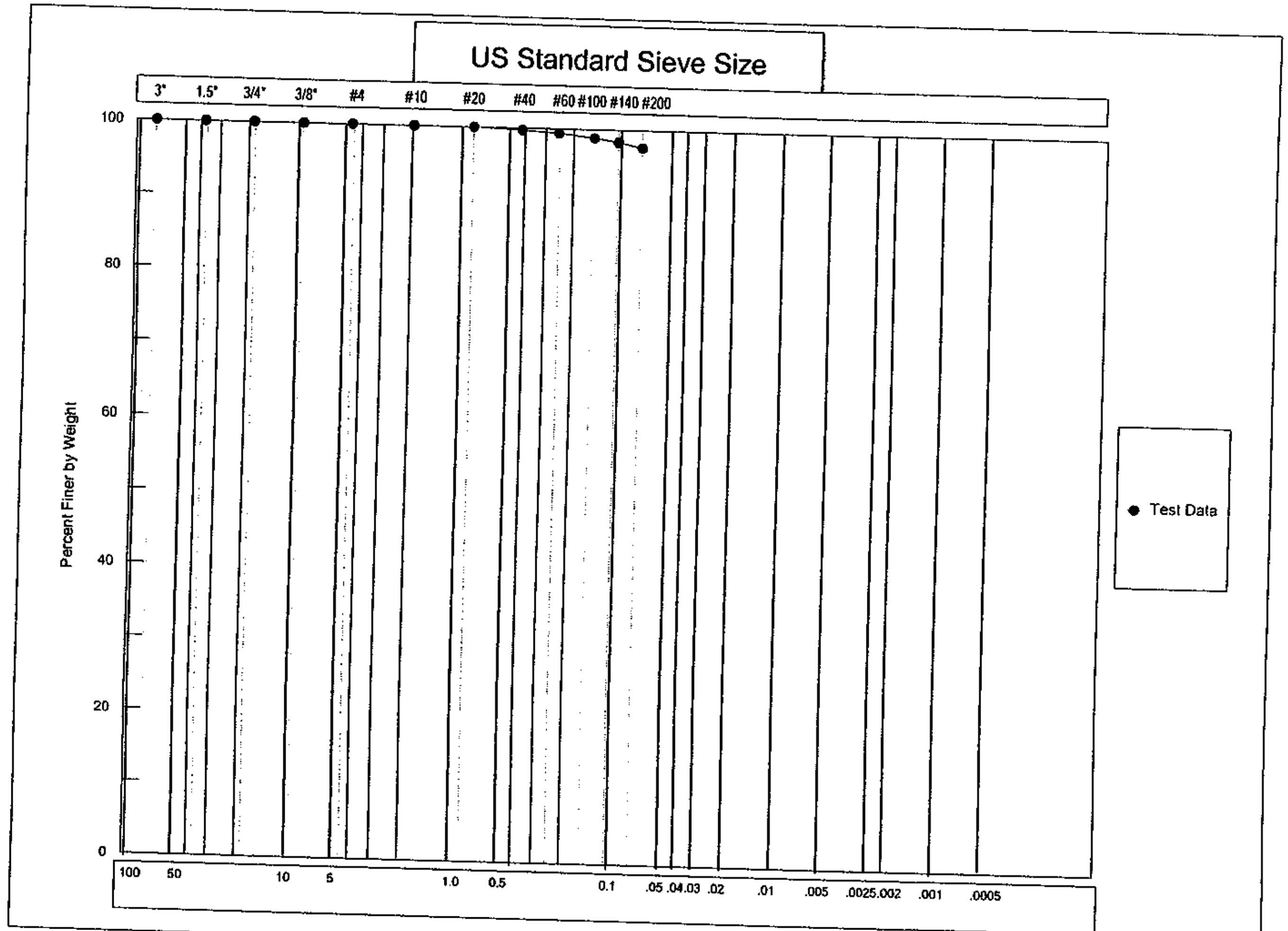
WASH SIEVE ANALYSIS

Wt. Total Sample Wet (g)	372.35
Weight of + #4 Before Washing (g)	20.14
Weight of + #4 After Washing (g)	0.00
Weight of - #4 Wet (g)	352.21
Weight of - #4 Dry (g)	358.50
Wt. Total Sample Dry (g)	358.50
Calc. Wt. "W" (g)	115.22
Calc. Mass + #4	0.00

Sieve Number (Size)	Pan Weight (g)	Indiv. Wt. + Pan (g)	Indiv. Wt. Retain.	Cum. Wt. Retain.	Cum. % Retain.	% Finer By Wt.
3"	0.00	0.00	0.00	0.00	0.0	100.0
1 1/2"	0.00	0.00	0.00	0.00	0.0	100.0
3/4"	0.00	0.00	0.00	0.00	0.0	100.0
3/8"	0.00	0.00	0.00	0.00	0.0	100.0
#4	0.00	0.00	0.00	0.00	0.0	100.0
#10	0.00	0.00	0.00	0.00	0.0	100.0
#20	3.70	3.70	0.00	0.00	0.0	100.0
#40	3.72	4.05	0.33	0.33	0.3	99.7
#60	3.69	4.12	0.43	0.76	0.7	99.3
#100	3.70	4.34	0.64	1.40	1.2	98.8
#140	3.69	4.32	0.63	2.03	1.8	98.2
#200	3.69	4.51	0.82	2.85	2.5	97.5

Data entered by: SR Date: 07/14/2006
Data checked by: RS Date: 7/14/06
FileName: GDM0R115

ADVANCED TERRA TESTING, INC.



COBBLES	GRAVEL		SAND			SILT OR CLAY	
	COARSE	FINE	CRS	MEDIUM	FINE		USCS

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			WENTWORTH

Client: Goodson & Associates Boring No.: BR-1
Job Number: 2014-104 Depth: 15'
Classification: **Classification Not Performed**

Sample No.:

Advanced Terra Testing, Inc.

MECHANICAL ANALYSIS
WITH HYDROMETER
ASTM D 422

MECHANICAL ANALYSIS - SIEVE TEST DATA
ASTM D 422

CLIENT Goodson & Associates

JOB NO. 2014-104

BORING NO.	BR-1	SAMPLED	
DEPTH	42.0-42.2	DATE TESTED	07-12-06 WAR
SAMPLE NO.		WASH SIEVE	Yes
SOIL DESCR.		DRY SIEVE	No
LOCATION	4th Street Bridge SH 96		

MOISTURE DATA

HYGROSCOPIC	Yes
NATURAL	No
Wt. Wet Soil & Pan (g)	51.69
Wt. Dry Soil & Pan (g)	50.93
Wt. Lost Moisture (g)	0.76
Wt. of Pan Only (g)	3.80
Wt. of Dry Soil (g)	47.13
Moisture Content %	1.6
Wt. Hydrom. Sample Wet (g)	57.62
Wt. Hydrom. Sample Dry (g)	56.71

WASH SIEVE ANALYSIS

Wt. Total Sample Wet (g)	57.62
Weight of + #10 Before Washing (g)	0.00
Weight of + #10 After Washing (g)	0.00
Weight of - #10 Wet (g)	57.62
Weight of - #10 Dry (g)	56.71
Wt. Total Sample Dry (g)	56.71
Calc. Wt. "W" (g)	56.71
Calc. Mass + #10	0.00

Sieve Number (Size)	Pan Weight (g)	Indiv. Wt. + Pan (g)	Indiv. Wt. Retain.	Cum. Wt. Retain.	Cum. % Retain.	% Finer By Wt.
3"	0.00	0.00	0.00	0.00	0.0	100.0
1 1/2"	0.00	0.00	0.00	0.00	0.0	100.0
3/4"	0.00	0.00	0.00	0.00	0.0	100.0
3/8"	0.00	0.00	0.00	0.00	0.0	100.0
#4	0.00	0.00	0.00	0.00	0.0	100.0
#10	0.00	0.00	0.00	0.00	0.0	100.0
#20	2.35	2.35	0.00	0.00	0.0	100.0
#40	2.28	2.30	0.02	0.02	0.0	100.0
#60	2.37	2.38	0.01	0.03	0.1	99.9
#100	2.35	2.38	0.03	0.06	0.1	99.9
#200	2.28	11.78	9.50	9.56	16.9	83.1

Data entered by: SR Date: 07/14/2006
Data checked by: RS Date: 7/14/06
FileName: GDH0BR1

ADVANCED TERRA TESTING, INC.

HYDROMETER ANALYSIS - SEDIMENTATION DATA
ASTM D 422

CLIENT	Goodson & Associates	JOB NO.	2014-104
BORING NO.	BR-1	SAMPLED	
DEPTH	42.0-42.2	DATE TESTED	07-12-06 WAR
SAMPLE NO.		WASH SIEVE	Yes
SOIL DESCR.		DRY SIEVE	No
LOCATION	4th Street Bridge SH 96		

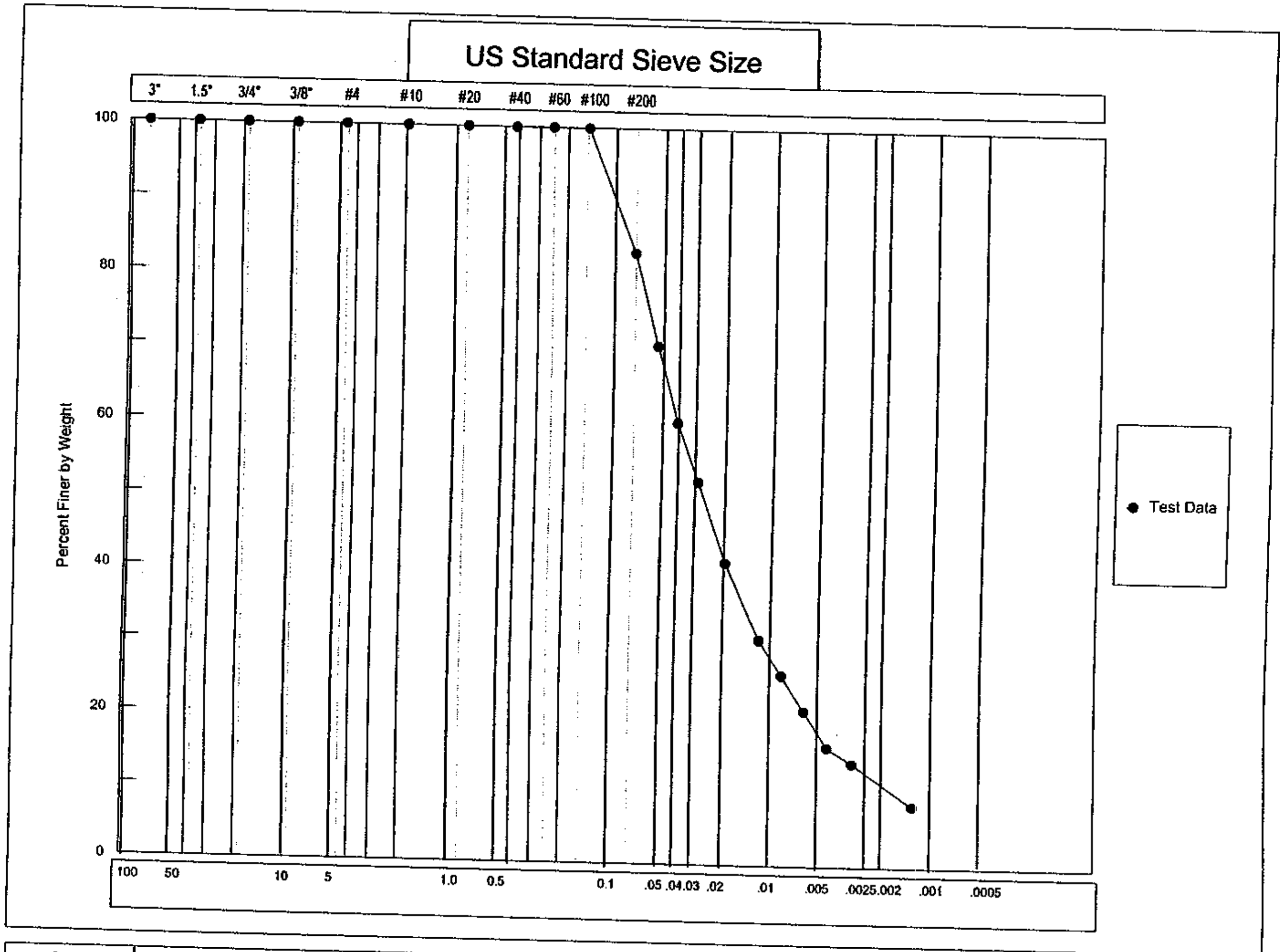
Hydrometer #	ASTM 152 H	Temp., Deg. C	25.8
Sp. Gr. of Soil	2.65	Temp. Coef. K	0.01275
Value of "alpha"	1.00	Wt. Dry Sample "W"	56.709
Deflocculant	Sodium Hexametaphosphate	% of Total Sample	100.0
Defloc. Corr'n	4.3		
Meniscus Corr'n	-1.0		

T Elapsed Time (min)	Hydrometer Reading Original	Corrected "R"	100Ra/W	% Total Sample	Effective Depth L	Grain Diameter (mm)
0.0	--	--	--	--	--	--
0.5	45.25	40.00	70.5	70.5	8.87	0.0537
1.0	39.50	34.25	60.4	60.4	9.81	0.0399
2.0	35.00	29.75	52.5	52.5	10.55	0.0293
5.0	28.75	23.50	41.4	41.4	11.58	0.0194
15.0	22.75	17.50	30.9	30.9	12.56	0.0117
30.0	20.00	14.75	26.0	26.0	13.01	0.0084
60.0	17.25	12.00	21.2	21.2	13.46	0.0060
120.0	14.50	9.25	16.3	16.3	13.91	0.0043
250.0	13.25	8.00	14.1	14.1	14.12	0.0030
1440.0	10.00	4.75	8.4	8.4	14.65	0.0013

Grain Diameter = K*(SQRT(L/T))

Data entered by: SR Date: 07/14/2006
Data checked by: RL Date: 7/14/06
FileName: GDH0BR1

ADVANCED TERRA TESTING, INC.



COBBLES	GRAVEL		SAND			SILT OR CLAY	USCS
	COARSE	FINE	CRS	MEDIUM	FINE		

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	WENTWORTH
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			

Client: Goodson & Associates Boring No.: BR-1
Job Number: 2014-104 Depth: 42.0-42.2
Classification: **Classification Not Performed**

Sample No.:

Advanced Terra Testing, Inc.

MECHANICAL ANALYSIS - SIEVE TEST DATA
ASTM D 422

CLIENT Goodson & associates

JOB NO. 2014-104

BORING NO.	BR-2
DEPTH	75.5-76.2
SAMPLE NO.	2" Bentonite Seam
SOIL DESCR.	
LOCATION	4th Street Bridge SH 96

SAMPLED	
DATE TESTED	07-10-06 WAR
WASH SIEVE	Yes
DRY SIEVE	No

MOISTURE DATA

HYGROSCOPIC	Yes
NATURAL	No

Wt. Wet Soil & Pan (g)	36.80
Wt. Dry Soil & Pan (g)	35.82
Wt. Lost Moisture (g)	0.98
Wt. of Pan Only (g)	3.72
Wt. of Dry Soil (g)	32.10
Moisture Content %	3.1
Wt. Hydrom. Sample Wet (g)	46.28
Wt. Hydrom. Sample Dry (g)	44.91

WASH SIEVE ANALYSIS

Wt. Total Sample Wet (g)	46.28
Weight of + #10 Before Washing (g)	0.00
Weight of + #10 After Washing (g)	0.00
Weight of - #10 Wet (g)	46.28
Weight of - #10 Dry (g)	44.91
Wt. Total Sample Dry (g)	44.91
Calc. Wt. "W" (g)	44.91
Calc. Mass + #10	0.00

Sieve Number (Size)	Pan Weight (g)	Indiv. Wt. + Pan (g)	Indiv. Wt. Retain.	Cum. Wt. Retain.	Cum. % Retain.	% Finer By Wt.
3"	0.00	0.00	0.00	0.00	0.0	100.0
1 1/2"	0.00	0.00	0.00	0.00	0.0	100.0
3/4"	0.00	0.00	0.00	0.00	0.0	100.0
3/8"	0.00	0.00	0.00	0.00	0.0	100.0
#4	0.00	0.00	0.00	0.00	0.0	100.0
#10	0.00	0.00	0.00	0.00	0.0	100.0
#20	2.36	2.38	0.02	0.02	0.0	100.0
#40	2.36	2.43	0.07	0.09	0.2	99.8
#60	2.35	2.40	0.05	0.14	0.3	99.7
#100	2.34	3.07	0.73	0.87	1.9	98.1
#200	2.29	9.53	7.24	8.11	18.1	81.9

Data entered by: SR Date: 07/12/2006
Data checked by: [initials] Date: 7/12/06
FileName: GDH07557

ADVANCED TERRA TESTING, INC.

HYDROMETER ANALYSIS - SEDIMENTATION DATA
ASTM D 422

CLIENT Goodson & associates

JOB NO. 2014-104

BORING NO. BR-2
DEPTH 75.5-76.2
SAMPLE NO. 2" Bentonite Seam
SOIL DESCR.
LOCATION 4th Street Bridge SH 96

SAMPLED
DATE TESTED 07-10-06 WAR
WASH SIEVE Yes
DRY SIEVE No

Hydrometer # ASTM 152 H
Sp. Gr. of Soil 2.65
Value of "alpha" 1.00
Deflocculant Sodium Hexametaphosphate
Defloc. Corr'n 4.5
Meniscus Corr'n -2.0

Temp., Deg. C 26.2
Temp. Coef. K 0.01269
Wt. Dry Sample "W" 44.912
% of Total Sample 100.0

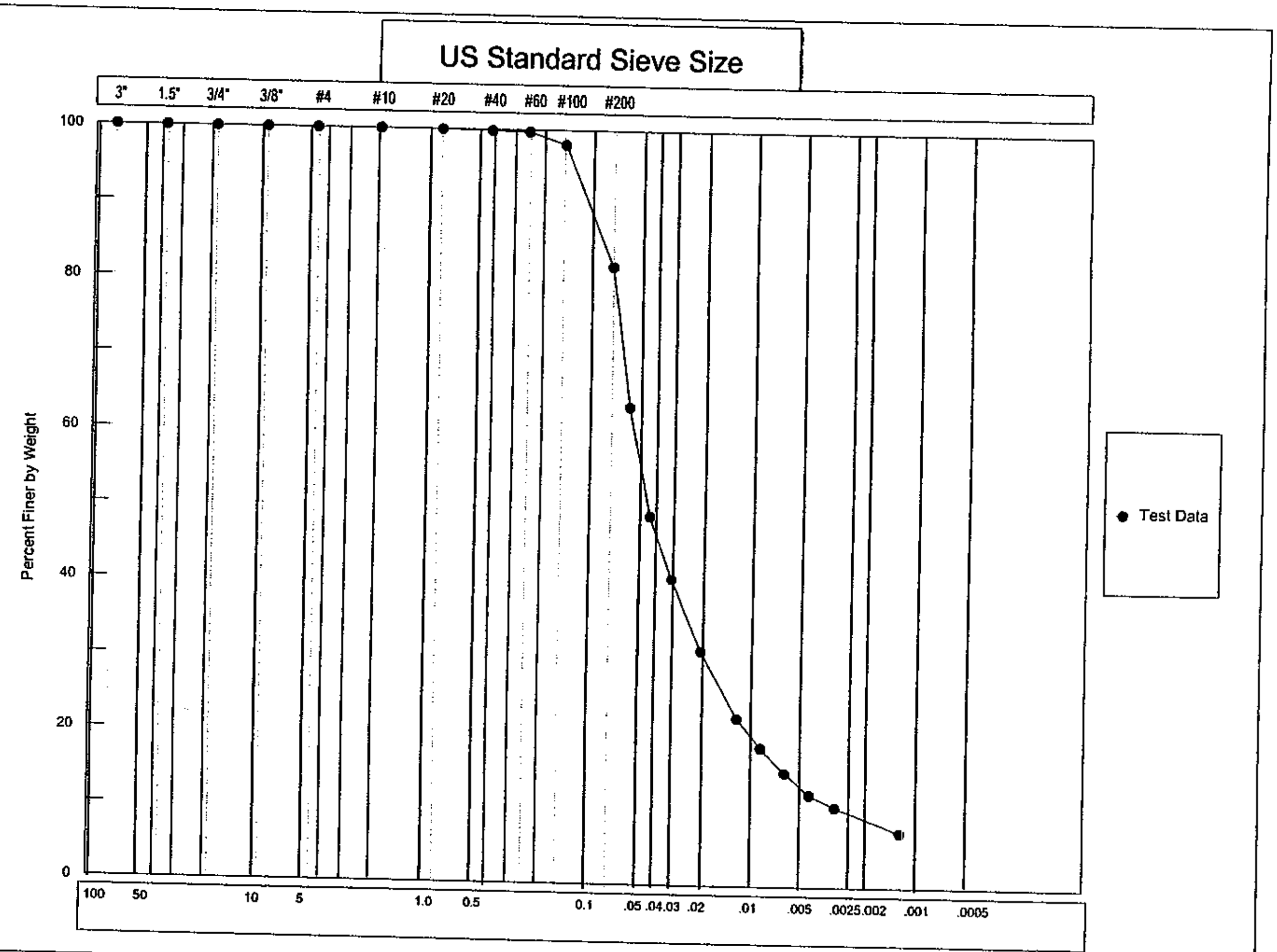
T Elapsed Time (min)	Hydrometer Reading Original	Corrected "R"	100Ra/W	% Total Sample	Effective Depth L	Grain Diameter (mm)
0.0	--	--	--	--	--	--
0.5	35.00	28.50	63.5	63.5	10.55	0.0583
1.0	28.50	22.00	49.0	49.0	11.62	0.0433
2.0	24.75	18.25	40.6	40.6	12.23	0.0314
5.0	20.50	14.00	31.2	31.2	12.93	0.0204
15.0	16.50	10.00	22.3	22.3	13.58	0.0121
30.0	14.75	8.25	18.4	18.4	13.87	0.0086
60.0	13.25	6.75	15.0	15.0	14.12	0.0062
120.0	12.00	5.50	12.2	12.2	14.32	0.0044
250.0	11.25	4.75	10.6	10.6	14.45	0.0031
1532.0	9.75	3.25	7.2	7.2	14.69	0.0012

Grain Diameter = K*(SQRT(L/T))

Data entered by: SR
Data checked by: *LS*
FileName: GDH07557

Date: 07/12/2006
Date: *7/12/06*

ADVANCED TERRA TESTING, INC.



COBBLES	GRAVEL		SAND			SILT OR CLAY	
	COARSE	FINE	CRS	MEDIUM	FINE		USCS

COBBLES TO BOULDERS	PEBBLE GRAVEL				SAND			SILT	CLAY	
	COARSE	MED	FINE	GRAN	COARSE	MED	FINE			WENTWORTH

Client: Goodson & associates Boring No.: BR-2
Job Number: 2014-104 Depth: 75.5-76.2
Classification: **Classification Not Performed**

Sample No.: 2" Bentonite Seam

Advanced Terra Testing, Inc.

DIRECT SHEAR TEST DATA
ASTM D 3080

DIRECT SHEAR TEST DATA
ASTM D 3080

CLIENT Goodson & Asscoiates

JOB NO. 2014-104

BORING NO.	BR-1	SAMPLED	
DEPTH	15'	TEST STARTED	07-06-06 DPM
SAMPLE NO.		TEST FINISHED	07-07-06 RS
POINT	A	CELL NUMBER	Geomatic
SOIL DESCR.		SATURATED TEST	Yes
LOCATION	4th Street Bridge SH 96	TEST TYPE	CD
NORMAL STRESS (psf)	3982		

MOISTURE/DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture (g)	93.8	100.9
Wt. Wet Soil & Pan (g)	101.9	109.1
Wt. Dry Soil & Pan (g)	90.9	90.9
Wt. Lost Moisture (g)	11.0	18.2
Wt. of Pan Only (g)	8.2	8.2
Wt. of Dry Soil (g)	82.7	82.7
Moisture Content %	13.3	21.9
Wet Density (pcf)	121.1	133.4
Dry Density (pcf)	106.9	109.4

Init. Diameter (in)	1.938
Init. Area (sq in)	2.950
Init. Height (in)	1.000
Vol. Bef. Consol. (cu ft)	0.00171
Final Height (in)	0.977
Vol. After Consol. (cu ft)	0.00167

Test Set Up: Continuous Increasing Deflection Control
Shear Speed: 0.00200 in/min

Notes & Comments:

Data File Name: GDDSBH1A.DAT

Normal Stress:	3982 psf	27.65 psi
Peak Stress:	4401 psf	30.56 psi
Ultimate Stress:	3151 psf	21.88 psi

Data entry by: SR Date: 07/10/2006
Data checked by: Rd Date: 7/10/06
FileName: GDDSR115

ADVANCED TERRA TESTING, INC.

DIRECT SHEAR TEST DATA
ASTM D 3080

CLIENT	Goodson & Asscoiates	JOB NO.	2014-104
BORING NO.	BR-1	SAMPLED	
DEPTH	15'	TEST STARTED	07-07-06 RS
SAMPLE NO.		TEST FINISHED	07-07-06 DPM
POINT	B	CELL NUMBER	Geomatic
SOIL DESCR.		SATURATED TEST	Yes
LOCATION	4th Street Bridge SH 96	TEST TYPE	CD
NORMAL STRESS (psf)	2010		

MOISTURE/DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture (g)	85.6	93.4
Wt. Wet Soil & Pan (g)	94.0	101.8
Wt. Dry Soil & Pan (g)	83.5	83.5
Wt. Lost Moisture (g)	10.5	18.3
Wt. of Pan Only (g)	8.4	8.4
Wt. of Dry Soil (g)	75.1	75.1
Moisture Content %	14.0	24.4
Wet Density (pcf)	110.5	124.2
Dry Density (pcf)	97.0	99.9

Init. Diameter (in)	1.938
Init. Area (sq in)	2.950
Init. Height (in)	1.000
Vol. Bef. Consol. (cu ft)	0.00171
Final Height (in)	0.971
Vol. After Consol. (cu ft)	0.00166

Test Set Up: Continuous Increasing Deflection Control
Shear Speed: 0.00200 in/min

Notes & Comments:

Data File Name: GDDSBH1B.DAT

Normal Stress:	2010 psf	13.96 psi
Peak Stress:	2573 psf	17.87 psi
Ultimate Stress:	2462 psf	17.10 psi

Data entry by: SR Date: 07/10/2006
Data checked by: Rd Date: 7/10/06
FileName: GDDSR115

ADVANCED TERRA TESTING, INC.

DIRECT SHEAR TEST DATA
ASTM D 3080

CLIENT	Goodson & Asscoiates	JOB NO.	2014-104
BORING NO.	BR-1	SAMPLED	
DEPTH	15'	TEST STARTED	07-06-06 RS
SAMPLE NO.		TEST FINISHED	07-07-06 RS
POINT	C	CELL NUMBER	Geomatic
SOIL DESCR.		SATURATED TEST	Yes
LOCATION	4th Street Bridge SH 96	TEST TYPE	CD
NORMAL STRESS (psf)	1005		

MOISTURE/DENSITY DATA	BEFORE TEST	AFTER TEST
Wt. Soil + Moisture (g)	92.5	101.4
Wt. Wet Soil & Pan (g)	100.6	109.6
Wt. Dry Soil & Pan (g)	90.8	90.8
Wt. Lost Moisture (g)	9.8	18.8
Wt. of Pan Only (g)	8.2	8.2
Wt. of Dry Soil (g)	82.7	82.7
Moisture Content %	11.9	22.7
Wet Density (pcf)	119.4	131.7
Dry Density (pcf)	106.8	107.3

Init. Diameter (in)	1.938
Init. Area (sq in)	2.950
Init. Height (in)	1.000
Vol. Bef. Consol. (cu ft)	0.00171
Final Height (in)	0.995
Vol. After Consol. (cu ft)	0.00170

Test Set Up: Continuous Increasing Deflection Control
Shear Speed: 0.00200 in/min

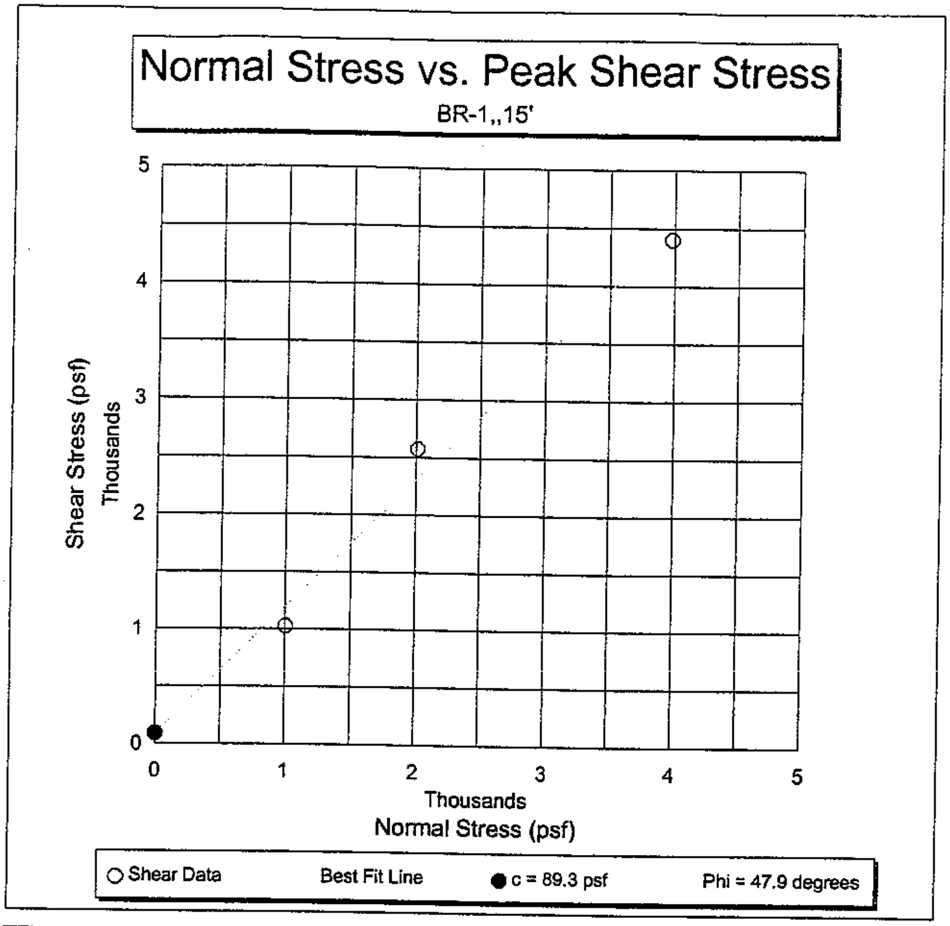
Notes & Comments:

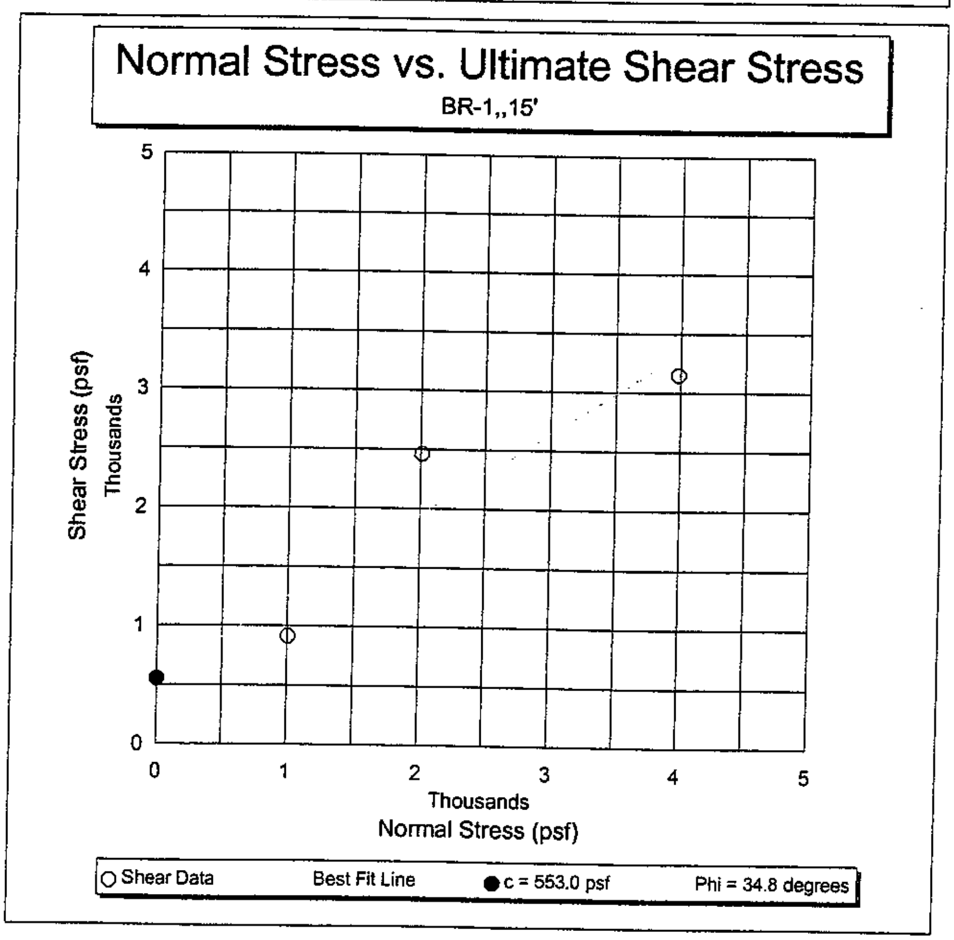
Data File Name: GDDSBH1C.DAT

Normal Stress:	1005 psf	6.98 psi
Peak Stress:	1026 psf	7.13 psi
Ultimate Stress:	914 psf	6.35 psi

Data entry by: SR Date: 07/10/2006
Data checked by: RS Date: 7/10/06
FileName: GDDSR115

ADVANCED TERRA TESTING, INC.





ADVANCED TERRA TESTING, INC.

GEOMATIC Direct Shear Data

File Name.....gddsbr1a.DAT
Client........Goodson & Associates
Job Number....201-104
Location......4th Street Bridge - SH 96
Date..........07-07-2006
Technician....RS
Sample Diameter (in)......1.938
Confining Pressure (psf)..3982
Number of Passes..........1
Shear Distance (in).......0.25
Shear Rate (in/min)........0020
Soil Type.................
Boring Number.............BR-1
Sample Number.............Pt. A
Depth (ft)................15'

SHEAR DEFLECTION (in)	SHEAR STRESS (lb/sqFT)	AXIAL DEFLECTION (in)
0.005	559	0.0000
0.010	970	-.0001
0.015	1324	0.0003
0.020	1585	0.0008
0.025	1977	0.0014
0.030	2350	0.0018
0.035	2704	0.0024
0.040	2984	0.0031
0.045	3151	0.0037
0.050	3357	0.0044
0.055	3543	0.0053
0.060	3674	0.0062
0.065	3841	0.0075
0.070	3953	0.0084
0.075	4065	0.0094
0.080	4196	0.0109
0.085	4270	0.0120
0.090	4382	0.0133
0.095	4196	0.0141
0.100	4028	0.0135
0.105	4065	0.0134
0.110	4009	0.0133
0.115	3953	0.0132
0.120	3879	0.0130
0.125	3543	0.0124
0.130	3543	0.0121
0.135	3524	0.0119
0.140	3468	0.0116
0.145	3375	0.0115
0.150	3338	0.0113
0.155	3301	0.0110
0.160	3301	0.0109
0.165	3301	0.0108
0.170	3263	0.0108
0.175	3263	0.0107

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?ass 1 Continued

SHEAR DEFLECTION (in)	SHEAR STRESS (lb/sqFT)	AXIAL DEFLECTION (in)
0.180	3301	0.0107
0.185	3282	0.0106
0.190	3282	0.0105
0.195	3263	0.0104
0.200	3226	0.0103
0.205	3245	0.0102
0.210	3263	0.0102
0.215	3263	0.0101
0.220	3263	0.0101
0.225	3245	0.0101
0.230	3245	0.0101
0.235	3226	0.0101
0.240	3189	0.0101
0.245	3170	0.0100
0.250	3151	0.0100

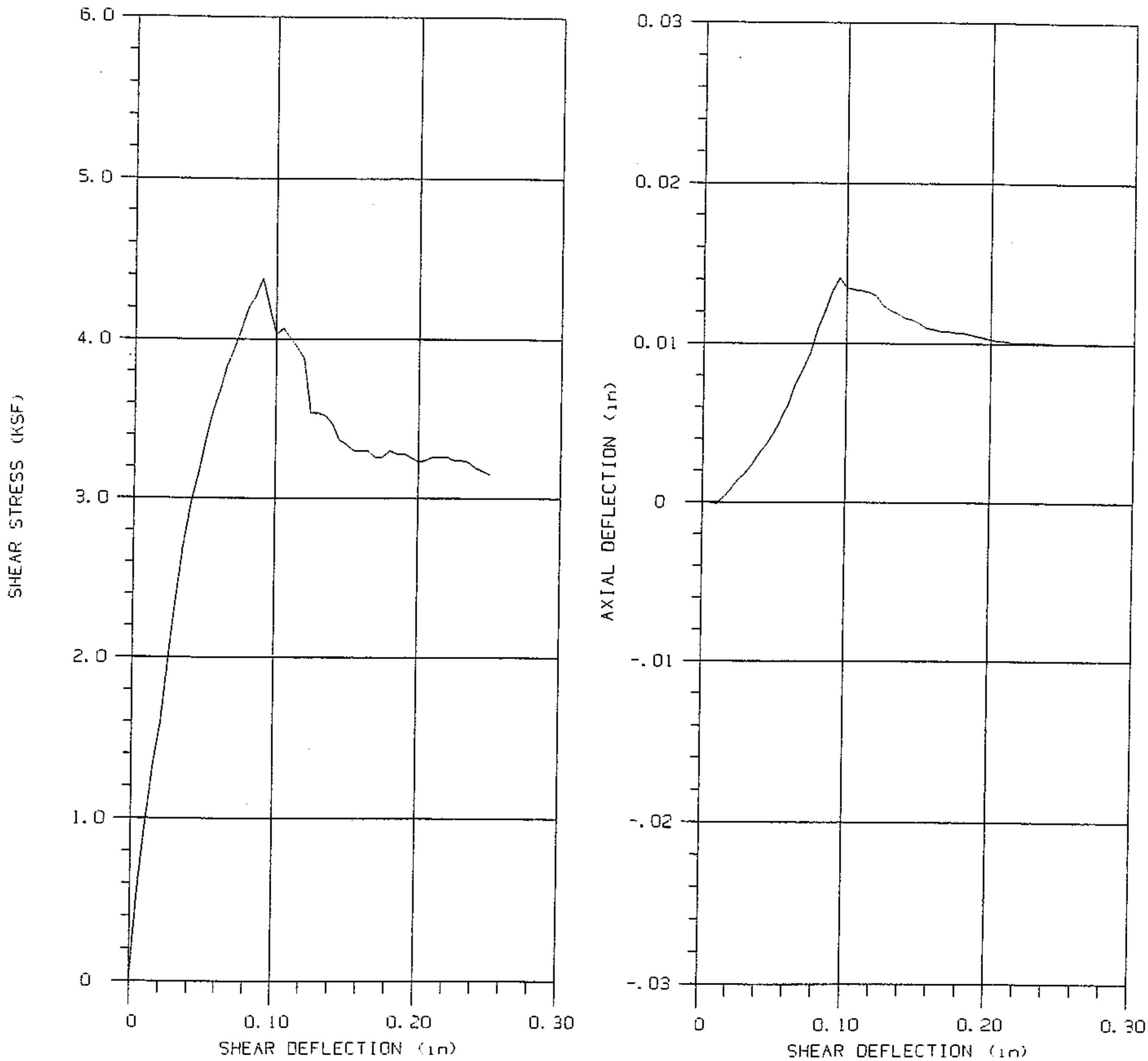
ass 1 completed with max load = 4401 psf at deflection of .0908 in

ADVANCED TERRA TESTING

Direct Shear Test

Client......... Goodson & Associates
Location....... 4th Street Bridge - SH 96

Soil Type..........		Job Number......	201-104
Depth...............	15 (ft)	Sample Number...	Pt. A
Confining Pressure..	3982 (psf)	Boring..........	BP-1
Shear Rate..........	.0020 (in/min)	File Name.......	gddsbr1a.DAT
Date Tested.........	07-07-2006	Tested By.......	RS



Pass Number and Line Type....	1 ———	2 ------	3 —— –	4 —·–·—
Maximum Shear Stress.........	4401	.0000	.0000	.0000
Shear Deflection @ Max Stress.	.0908	.0000	.0000	.0000

EOMATIC Direct Shear Data

'ile Name.....gddsbr1b.DAT
'lient........Goodson & Associates
'ob Number....201-104
.ocation......4th Street Bridge - SH 96
)ate..........07-07-2006
echnician....DPM
.ample Diameter (in)......1.938
'onfining Pressure (psf)..2010
umber of Passes..........1
hear Distance (in).......0.25
;hear Rate (in/min)........0020
'oil Type.................
oring Number.............BR-1
;ample Number.............Pt. B
)epth (ft)................15'

SHEAR)EFLECTION (in)	SHEAR STRESS (lb/sqFT)	AXIAL DEFLECTION (in)
0.005	578	-.0002
0.010	727	-.0013
0.015	876	-.0023
0.020	1026	-.0033
0.025	1156	-.0040
0.030	1287	-.0045
0.035	1361	-.0048
0.040	1473	-.0049
0.045	1585	-.0049
0.050	1660	-.0049
0.055	1716	-.0049
0.060	1772	-.0049
0.065	1846	-.0048
0.070	1883	-.0046
0.075	1939	-.0045
0.080	1995	-.0041
0.085	2051	-.0039
0.090	2089	-.0035
0.095	2144	-.0033
0.100	2182	-.0030
0.105	2219	-.0025
0.110	2256	-.0025
0.115	2294	-.0023
0.120	2312	-.0023
0.125	2070	-.0032
0.130	1958	-.0039
0.135	2144	-.0039
0.140	2331	-.0033
0.145	2406	-.0027
0.150	2406	-.0026
0.155	2443	-.0025
0.160	2462	-.0024
0.165	2499	-.0022
0.170	2517	-.0018
0.175	2517	-.0018

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Pass 1 Continued

SHEAR DEFLECTION (in)	SHEAR STRESS (lb/sqFT)	AXIAL DEFLECTION (in)
0.180	2517	-.0016
0.185	2555	-.0011
0.190	2555	-.0008
0.195	2555	-.0005
0.200	2573	-.0001
0.205	2555	0.0001
0.210	2536	0.0004
0.215	2536	0.0006
0.220	2517	0.0008
0.225	2499	0.0008
0.230	2499	0.0011
0.235	2480	0.0014
0.240	2462	0.0014
0.245	2443	0.0014
0.250	2462	0.0014

Pass 1 completed with max load = 2573 psf at deflection of .1906 in

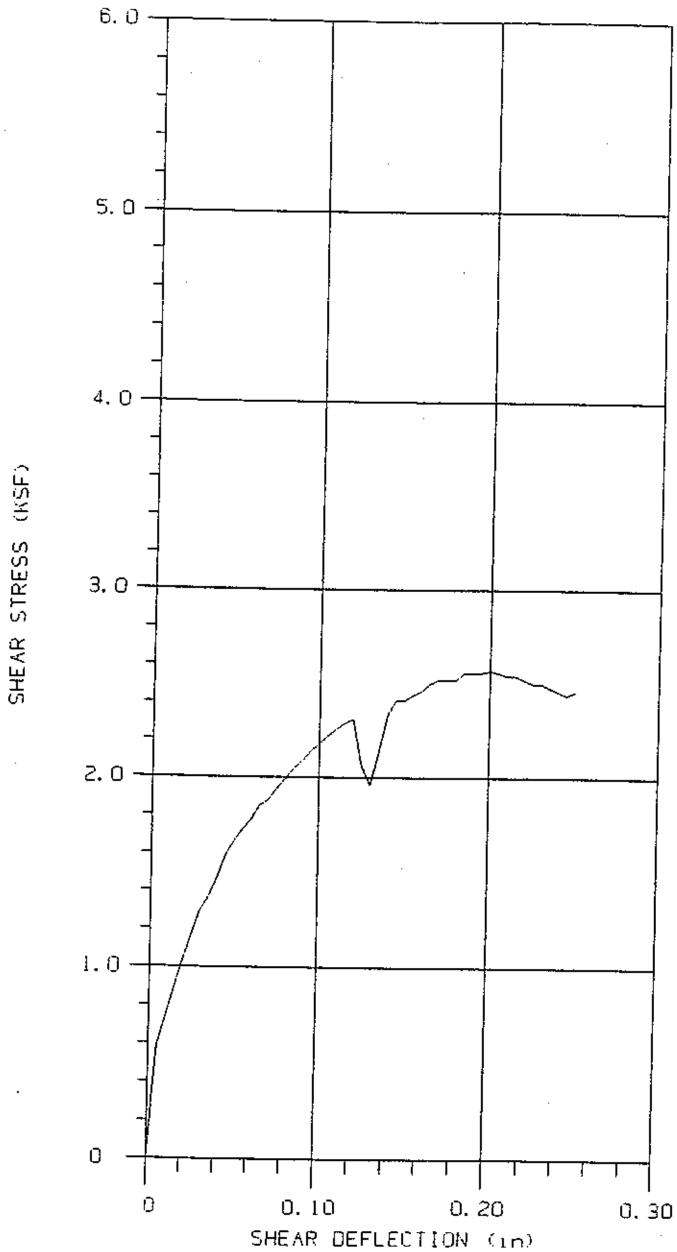
ADVANCED TERRA TESTING

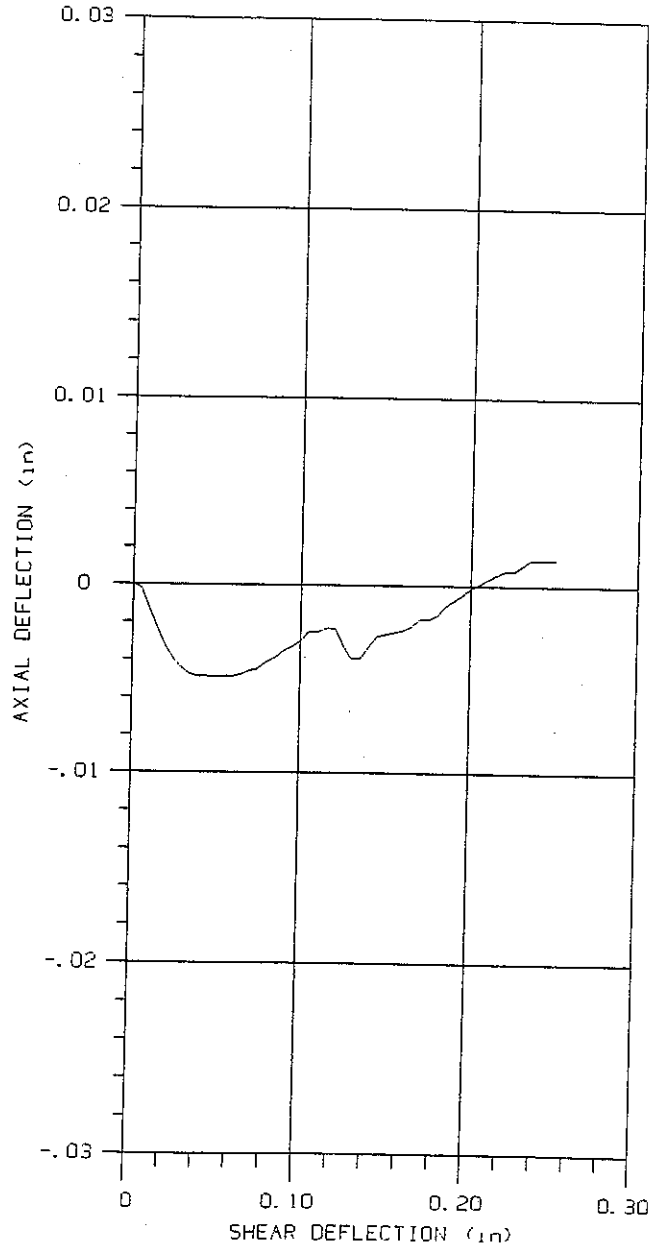
Direct Shear Test

Client......... Goodson & Associates
Location....... 4th Street Bridge - SH 96

Soil Type..........
Depth.............. 15 (ft)
Confining Pressure.. 2010 (psf)
Shear Rate......... .0020 (in/min)
Date Tested........ 07-07-2006

Job Number...... 201-104
Sample Number... Pt. B
Boring.......... BR-1
File Name....... gddsbrib.DAT
Tested By....... DPM





Pass Number and Line Type....	1 ———	2 ------	3 —— –	4 ——-·-—
Maximum Shear Stress.........	2573	.0000	.0000	.0000
Shear Deflection @ Max Stress.	.1906	.0000	.0000	.0000

GEOMATIC Direct Shear Data

File Name.....gddsbr1c.DAT
Client........Goodson & Associates
Job Number....201-104
Location......4th Street Bridge - SH 96
Date..........07-07-2006
Technician....RS
Sample Diameter (in)......1.938
Confining Pressure (psf)..1005
Number of Passes..........1
Shear Distance (in).......0.25
Shear Rate (in/min)........0020
Soil Type.................
Boring Number.............BR-1
Sample Number.............Pt. C
Depth (ft)................15'

SHEAR DEFLECTION (in)	SHEAR STRESS (lb/sqFT)	AXIAL DEFLECTION (in)
0.005	93	-.0002
0.010	168	0.0003
0.015	280	0.0008
0.020	466	0.0016
0.025	615	0.0021
0.030	727	0.0025
0.035	821	0.0029
0.040	876	0.0035
0.045	914	0.0039
0.050	932	0.0044
0.055	951	0.0051
0.060	970	0.0055
0.065	970	0.0061
0.070	988	0.0067
0.075	988	0.0072
0.080	988	0.0077
0.085	1007	0.0084
0.090	1007	0.0090
0.095	1007	0.0095
0.100	1026	0.0101
0.105	1007	0.0108
0.110	1007	0.0115
0.115	1007	0.0122
0.120	1007	0.0127
0.125	1007	0.0133
0.130	1007	0.0141
0.135	970	0.0148
0.140	970	0.0156
0.145	951	0.0162
0.150	951	0.0169
0.155	951	0.0176
0.160	951	0.0183
0.165	951	0.0189
0.170	932	0.0198
0.175	932	0.0205

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Pass 1 Continued

SHEAR DEFLECTION (in)	SHEAR STRESS (lb/sqFT)	AXIAL DEFLECTION (in)
0.180	932	0.0213
0.185	932	0.0220
0.190	914	0.0228
0.195	914	0.0236
0.200	914	0.0244
0.205	932	0.0251
0.210	932	0.0258
0.215	932	0.0265
0.220	932	0.0273
0.225	932	0.0280
0.230	932	0.0287
0.235	914	0.0295
0.240	914	0.0302
0.245	914	0.0309
0.250	914	0.0317

Pass 1 completed with max load = 1026 psf at deflection of .0933 in

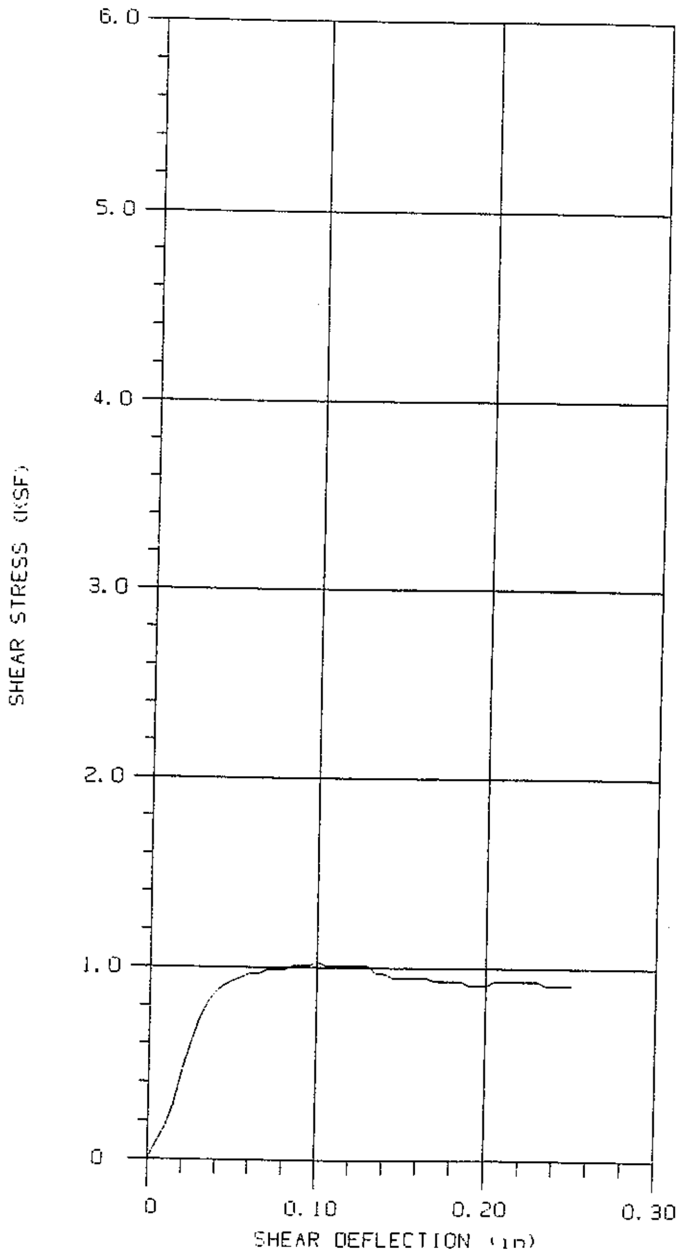
ADVANCED TERRA TESTING

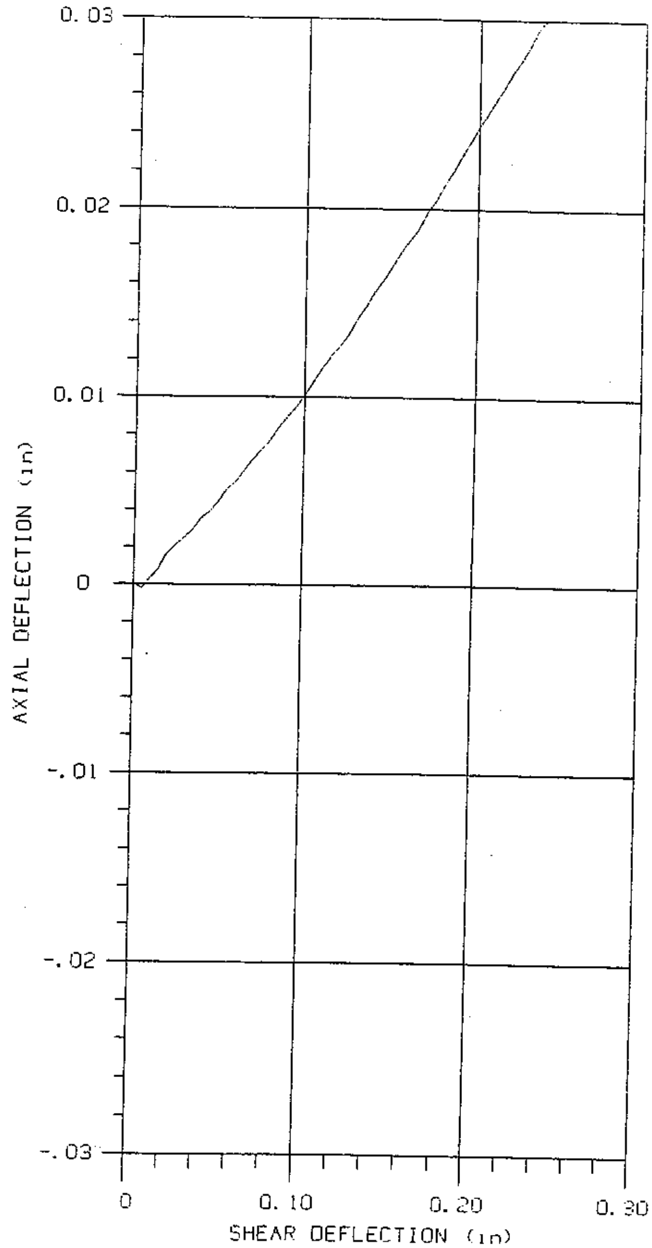
Direct Shear Test

Client.......... Goodson & Associates
Location........ 4th Street Bridge - SH 96

Soil Type...........
Depth.............. 15 (ft)
Confining Pressure.. 1005 (psf)
Shear Rate.......... .0020 (in/min)
Date Tested......... 07-07-2006

Job Number...... 201-104
Sample Number... Pt. C
Boring.......... BR-1
File Name....... gddsbr1c.DAT
Tested By....... RS





Pass Number and Line Type....	1 ———	2 ------	3 —— -	4 —-·--
Maximum Shear Stress.........	1026	.0000	.0000	.0000
Shear Deflection @ Max Stress.	.0933	.0000	.0000	.0000

SPLITTING TENSILE STRENGTH
By Method of Brazilian Disk
ASTM D 3967

SPLITTING TENSILE STRENGTH
By Method of Brazilian Disk
ASTM D 3967

CLIENT: Goodson & Associates, Inc.

JOB NO.: 2014-104

LOCATION:

DATE TESTED: 6/30/06 HN

PROJECT: 4th Street Bridge

Specimen ID Boring, Depth(ft.)	Diameter (in.)	Length (in.)	Mass (gms)	Wet Density (pcf)	Failure Load (lb)	Failure Type *	Splitting Tensile Strength (psi)
BR-3, 11.1-11.8	2.406	1.212	202.20	139.8	1,115	M	240
BR-3, 24.0-25.0	2.408	1.190	211.80	148.9	3,090	S	690

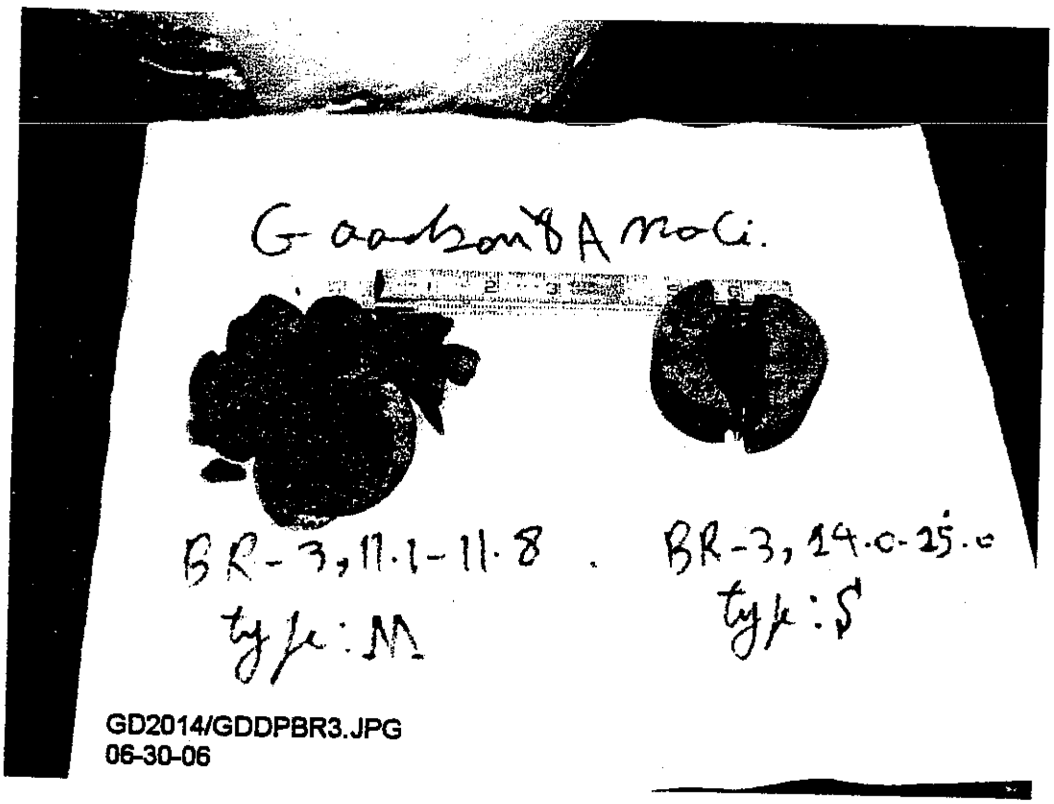
Notes and Comments:

Splitting Tensile Strength=2P/piLD.
P=Failure Load
pi = 3.1415926....
D = Sample Diameter
L = Sample Length

* Failure Type: S: Single Failure Plane, M: Multiple Failure Planes

Data Entered By: HN Date: 06/30/2006
Data Checked By: ω Date: 07/05/06
Filename: GDBRAZCP

ADVANCED TERRA TESTING,



GD2014/GDDPBR3.JPG
06-30-06

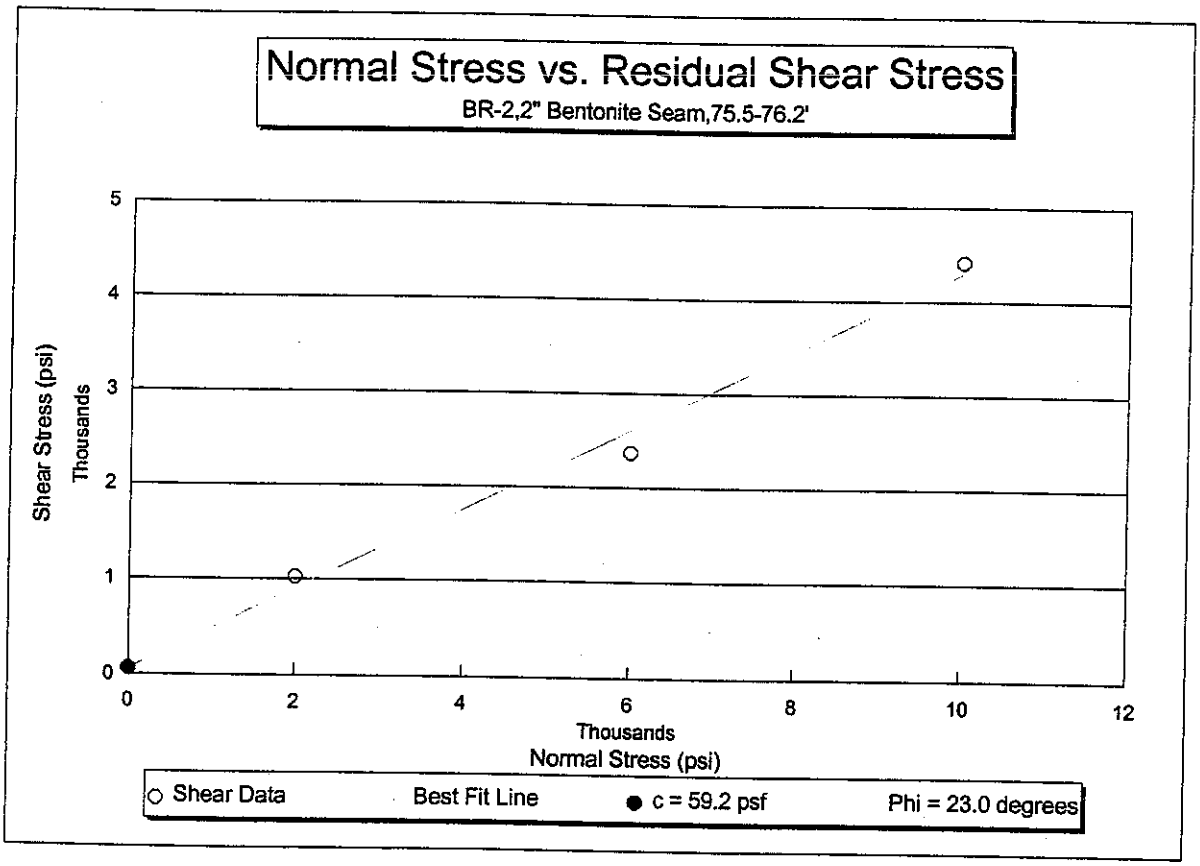
JOINT DIRECT SHEAR

ASTM D 3080 Modified

JOINT DIRECT SHEAR
Modified ASTM D 3080

CLIENT	Goodson & Associates	JOB NO.	2014-104
BORING NO.	BR-2	DATE SAMPLED	
DEPTH	75.5-76.2'	DATE TESTED	07-05-06 DPM
SAMPLE NO.	2" Bentonite Seam	LOCATION	4th Street Bridge SH 96
ROCK TYPE	Bentonite Seam	JOINT TYPE	Intact Rock

NORMAL STRESS (psf)	RESIDUAL SHEAR STRESS (psf)
10000	4418
6000	2369
2000	1025



Notes and Comments:

Data entered by: SR Date: 07/06/2006
Data checked by: DS Date: 7/10/06
File Name: GDDSBR27

ADVANCED TERRA TESTING, inc.

DIRECT SHEAR TEST DATA
ASTM D3080

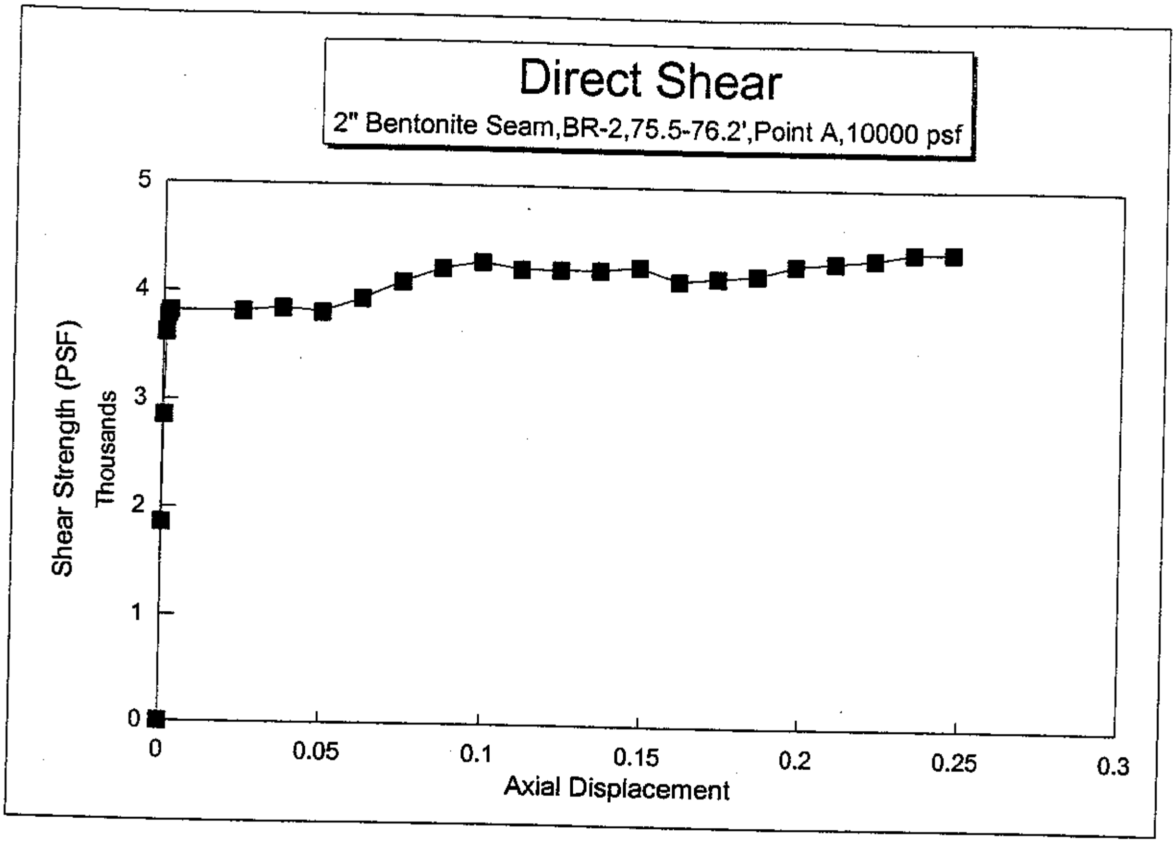
Client:	Goodson & Associates	Job Number:	2014-104
		Date Tested:	07-05-06 DPM
Boring:	BR-2		
Sample Number:	2" Bentonite Seam	Soil Description:	
Depth:	75.5-76.2'	Point:	A
Location:	4th Street Bridge SH 96	Normal Load	10000 PSF
		Peak Strength	4418 PSF

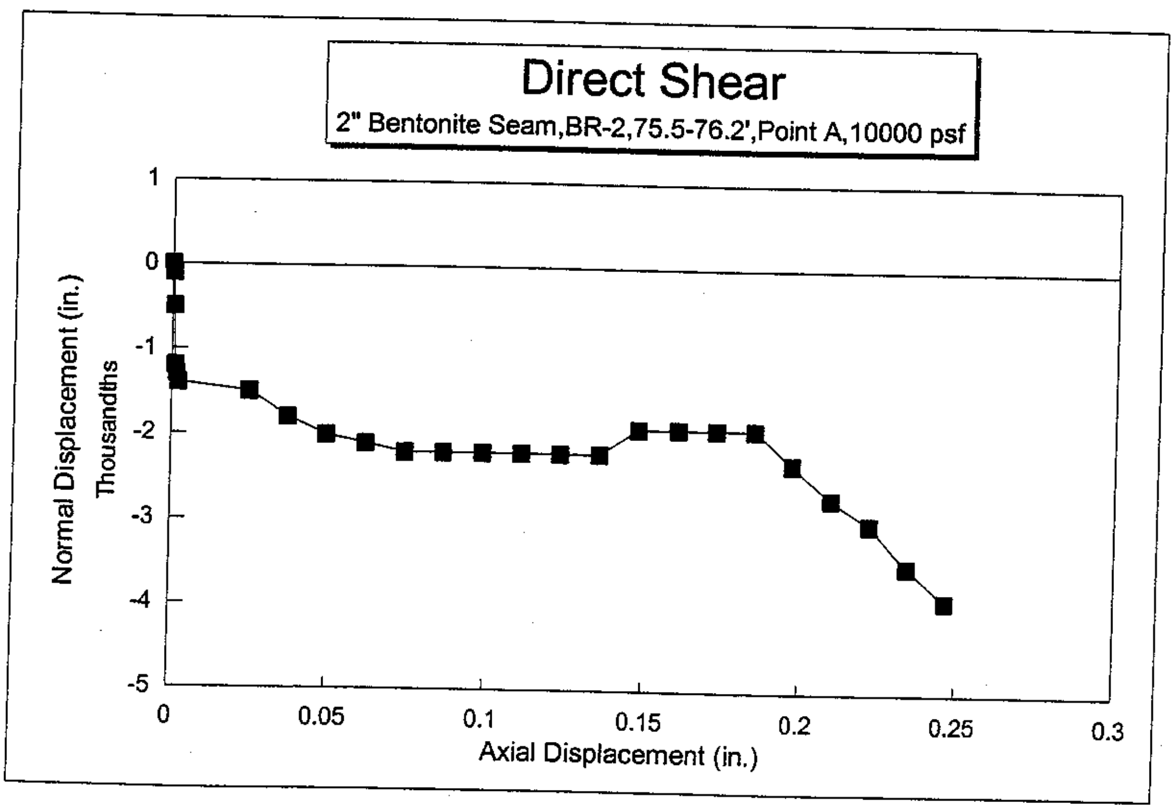
Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0002	58.0	1857	-0.0001
0.0005	89.0	2850	-0.0005
0.0010	113.0	3618	-0.0012
0.0015	118.0	3778	-0.0013
0.0020	119.0	3810	-0.0014
0.0247	119.0	3810	-0.0015
0.0370	120.0	3842	-0.0018
0.0493	119.0	3810	-0.0020
0.0616	123.0	3938	-0.0021
0.0740	128.0	4098	-0.0022
0.0863	132.0	4226	-0.0022
0.0986	134.0	4290	-0.0022
0.1109	132.0	4226	-0.0022
0.1233	132.0	4226	-0.0022
0.1356	132.0	4226	-0.0022
0.1479	133.0	4258	-0.0019
0.1602	129.0	4130	-0.0019
0.1726	130.0	4162	-0.0019
0.1849	131.0	4194	-0.0019
0.1972	134.0	4290	-0.0023
0.2095	135.0	4322	-0.0027
0.2219	136.0	4354	-0.0030
0.2342	138.0	4418	-0.0035
0.2465	138.0	4418	-0.0039

Data entry by: SR
Checked by: RL
FileName: GDDS2BS

Date: 07/06/2006
Date: 7/10/06

ADVANCED TERRA TESTING, INC


Direct Shear
2" Bentonite Seam,BR-2,75.5-76.2',Point A,10000 psf
Shear Strength (PSF)
Thousands
5
4
3
2
1
0
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement


Direct Shear
2" Bentonite Seam,BR-2,75.5-76.2',Point A,10000 psf
Normal Displacement (in.)
Thousandths
1
0
-1
-2
-3
-4
-5
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement (in.)

DIRECT SHEAR TEST DATA
ASTM D3080

Client: Goodson & Associates

Boring: BR-2
Sample Number: 2" Bentonite Seam
Depth: 75.5-76.2'
Location: 4th Street Bridge SH 96

Job Number: 2014-104
Date Tested: 07-05-06 DPM

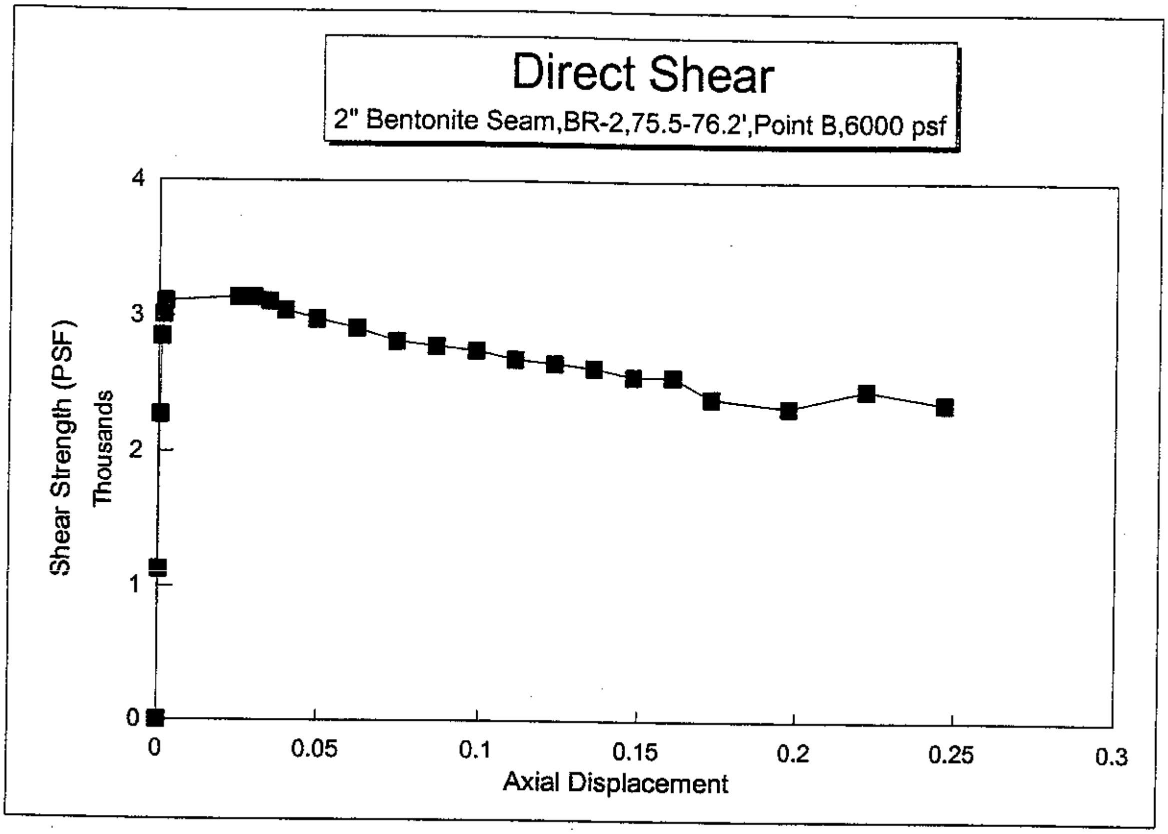
Soil Description:
Point: B
Normal Load 6000 PSF
Peak Strength 3138 PSF

Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0002	35.0	1121	-0.0001
0.0005	71.0	2273	-0.0007
0.0010	89.0	2850	-0.0008
0.0015	94.0	3010	-0.0008
0.0020	97.0	3106	-0.0006
0.0247	98.0	3138	-0.0001
0.0296	98.0	3138	0.0004
0.0345	97.0	3106	0.0008
0.0394	95.0	3042	0.0015
0.0493	93.0	2978	0.0026
0.0616	91.0	2914	0.0041
0.0740	88.0	2818	0.0059
0.0863	87.0	2786	0.0072
0.0986	86.0	2754	0.0090
0.1109	84.0	2689	0.0103
0.1233	83.0	2657	0.0119
0.1356	82.0	2625	0.0135
0.1479	80.0	2561	0.0151
0.1602	80.0	2561	0.0162
0.1726	75.0	2401	0.0172
0.1972	73.0	2337	0.0179
0.2219	77.0	2465	0.0181
0.2465	74.0	2369	0.0184

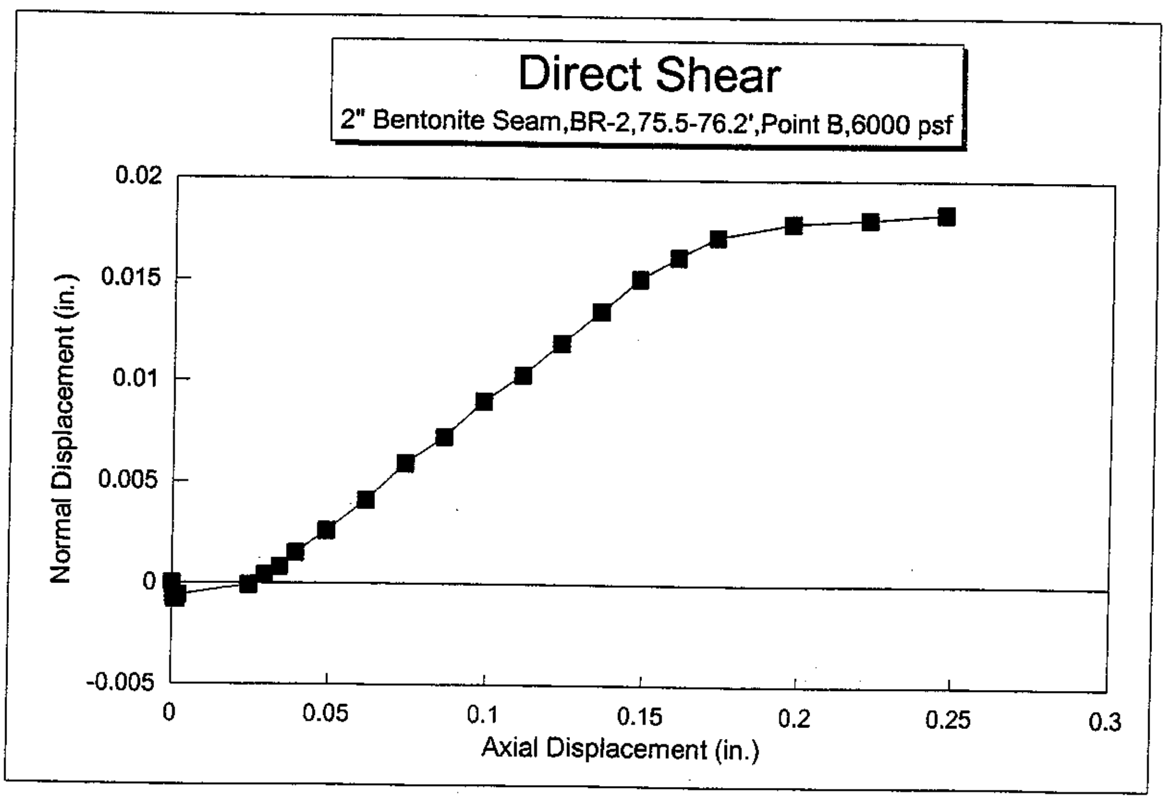
Data entry by: SR
Checked by: RB
FileName: GDDS2BSB

Date: 07/06/2006
Date: 7/10/06

ADVANCED TERRA TESTING, INC



Direct Shear
2" Bentonite Seam,BR-2,75.5-76.2',Point B,6000 psf
Shear Strength (PSF)
Thousands
Axial Displacement
0
1
2
3
4
0
0.05
0.1
0.15
0.2
0.25
0.3



Direct Shear
2" Bentonite Seam,BR-2,75.5-76.2',Point B,6000 psf
Normal Displacement (in.)
Axial Displacement (in.)
-0.005
0
0.005
0.01
0.015
0.02
0
0.05
0.1
0.15
0.2
0.25
0.3

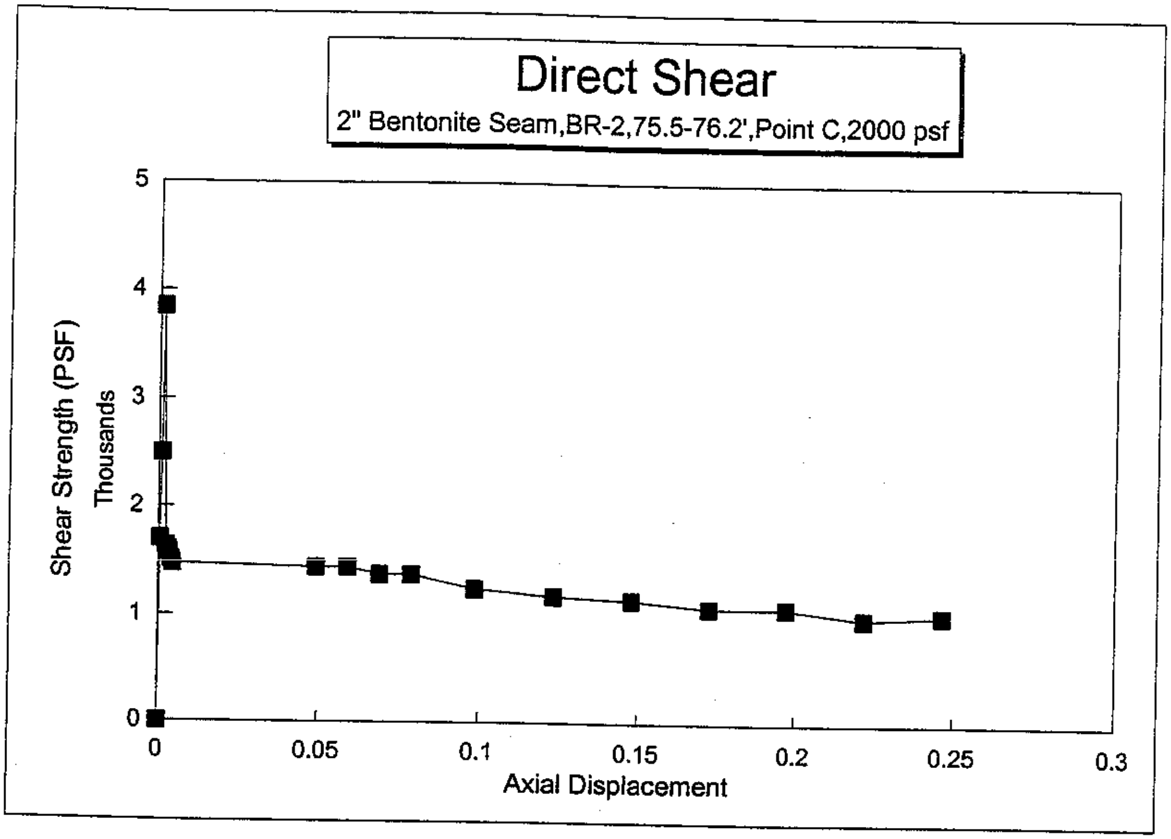
DIRECT SHEAR TEST DATA
ASTM D3080

Client:	Goodson & Associates	Job Number:	2014-104
		Date Tested:	07-05-06 DPM
Boring:	BR-2		
Sample Number:	2" Bentonite Seam	Soil Description:	
Depth:	75.5-76.2'	Point:	C
Location:	4th Street Bridge SH 96	Normal Load	2000 PSF
		Peak Strength	3842 PSF

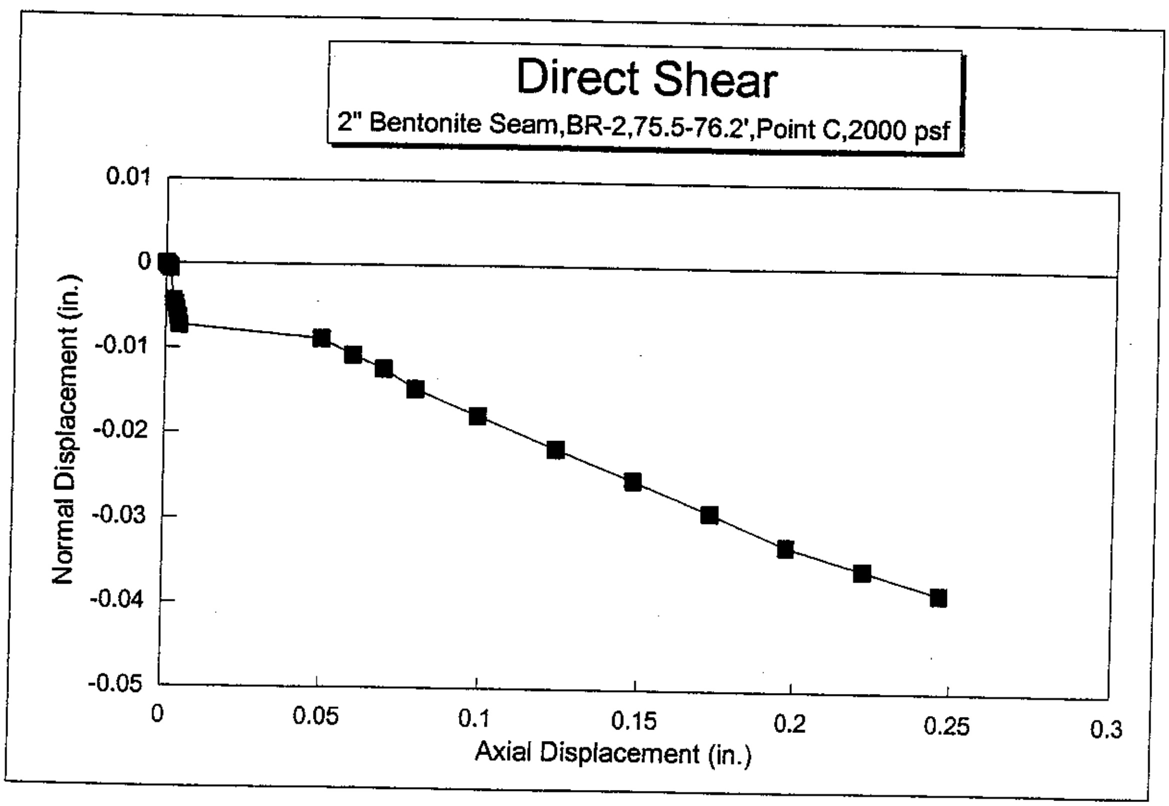
Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0005	53.0	1697	-0.0002
0.0010	78.0	2497	-0.0004
0.0015	120.0	3842	-0.0006
0.0025	51.0	1633	-0.0044
0.0030	50.0	1601	-0.0048
0.0035	49.0	1569	-0.0055
0.0039	47.0	1505	-0.0063
0.0044	46.0	1473	-0.0073
0.0493	45.0	1441	-0.0088
0.0592	45.0	1441	-0.0107
0.0690	43.0	1377	-0.0123
0.0789	43.0	1377	-0.0147
0.0986	39.0	1249	-0.0178
0.1233	37.0	1185	-0.0216
0.1479	36.0	1153	-0.0252
0.1726	34.0	1089	-0.0289
0.1972	34.0	1089	-0.0328
0.2219	31.0	993	-0.0355
0.2465	32.0	1025	-0.0383

Data entry by: SR Date: 07/06/2006
Checked by: ns Date: 7/10/06
FileName: GDDS2BSC

ADVANCED TERRA TESTING, INC



Direct Shear
2" Bentonite Seam,BR-2,75.5-76.2',Point C,2000 psf
Shear Strength (PSF)
Thousands
5
4
3
2
1
0
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement

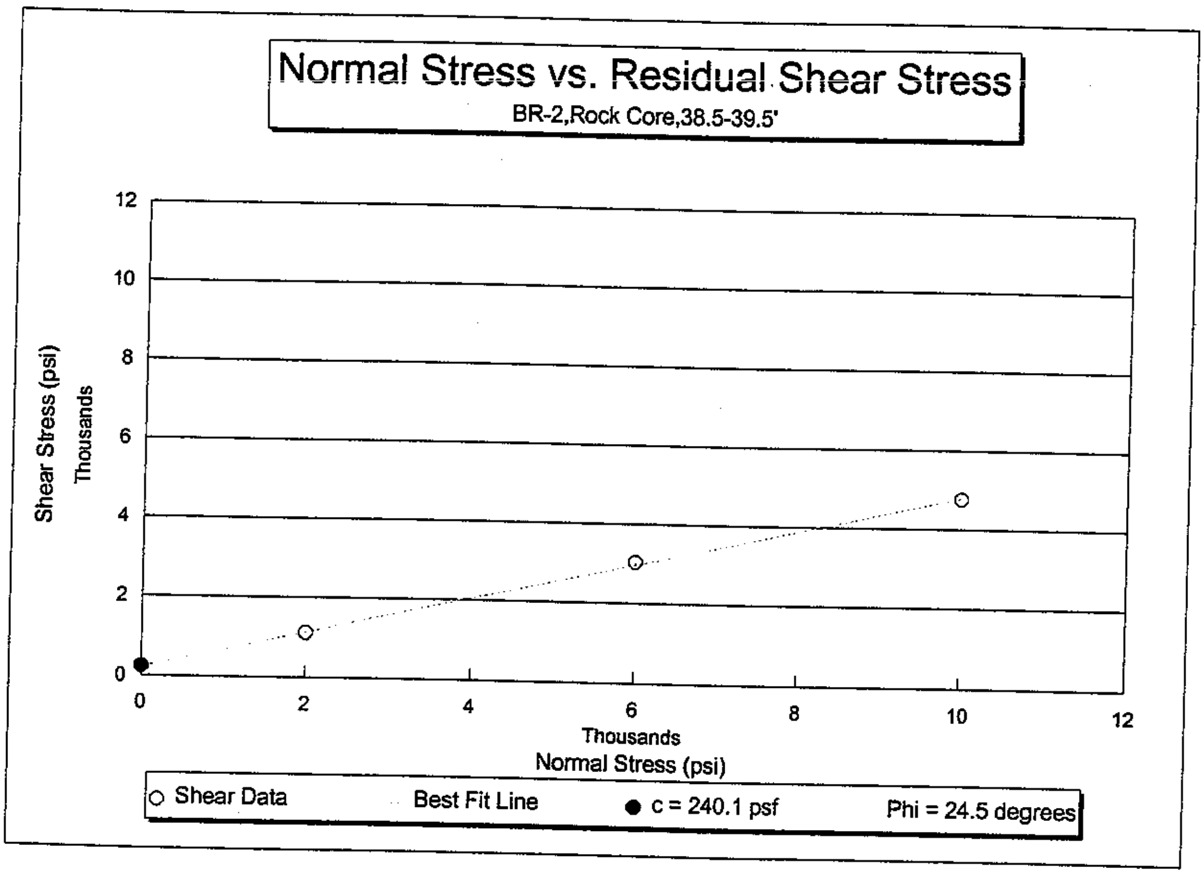


Direct Shear
2" Bentonite Seam,BR-2,75.5-76.2',Point C,2000 psf
Normal Displacement (in.)
0.01
0
-0.01
-0.02
-0.03
-0.04
-0.05
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement (in.)

JOINT DIRECT SHEAR
ASTM D 3080/MODIFIED

CLIENT	Godson & Associates	JOB NO.	2014-104
BORING NO.	BR-2	DATE SAMPLED	
DEPTH	38.5-39.5'	DATE TESTED	06-23,26&27-06 DPM/KB
SAMPLE NO.	Rock Core	LOCATION	4th Street Bridge, SH 96
ROCK TYPE		JOINT TYPE	Saw Cut

NORMAL STRESS (psf)	RESIDUAL SHEAR STRESS (psf)
10000	4767
6000	3039
2000	1120



Notes and Comments:

Data entered by: SR Date: 06/27/2006
Data checked by: RS Date: 7/17/06
File Name: GDDSBR2

ADVANCED TERRA TESTING, inc.

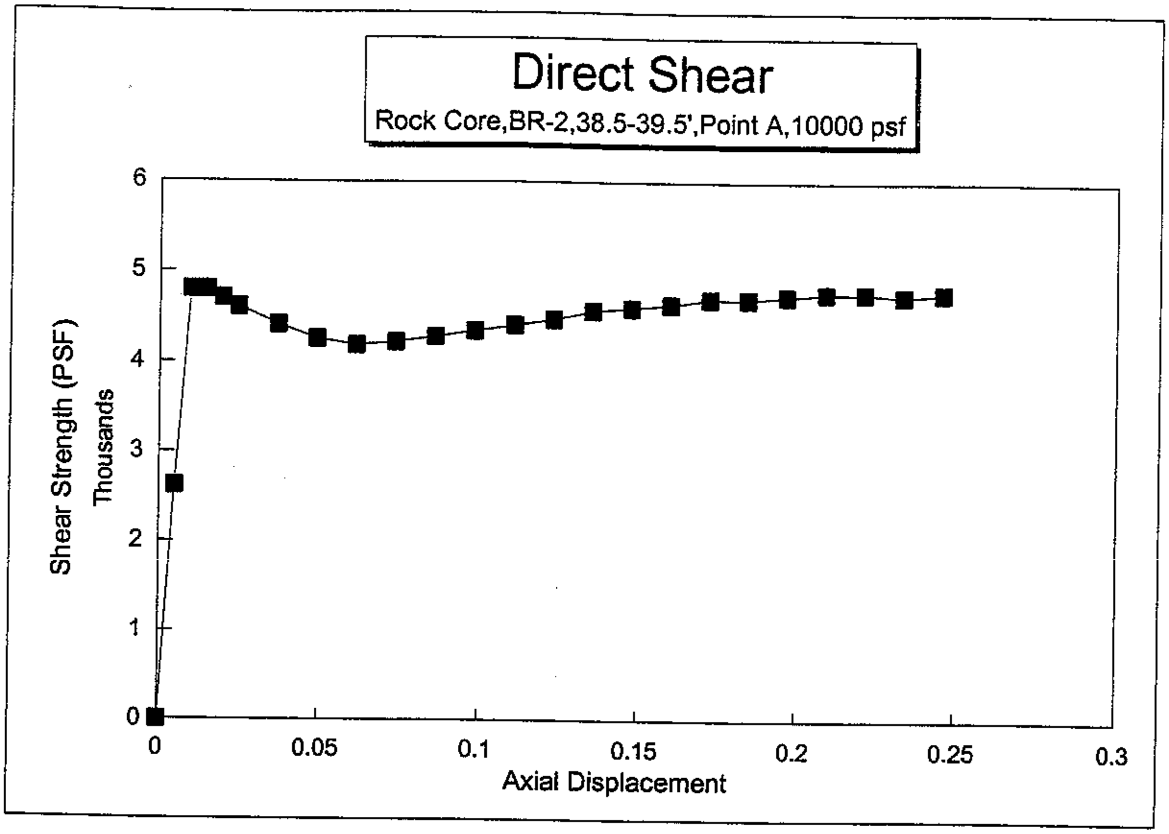
DIRECT SHEAR TEST DATA
ASTM D3080

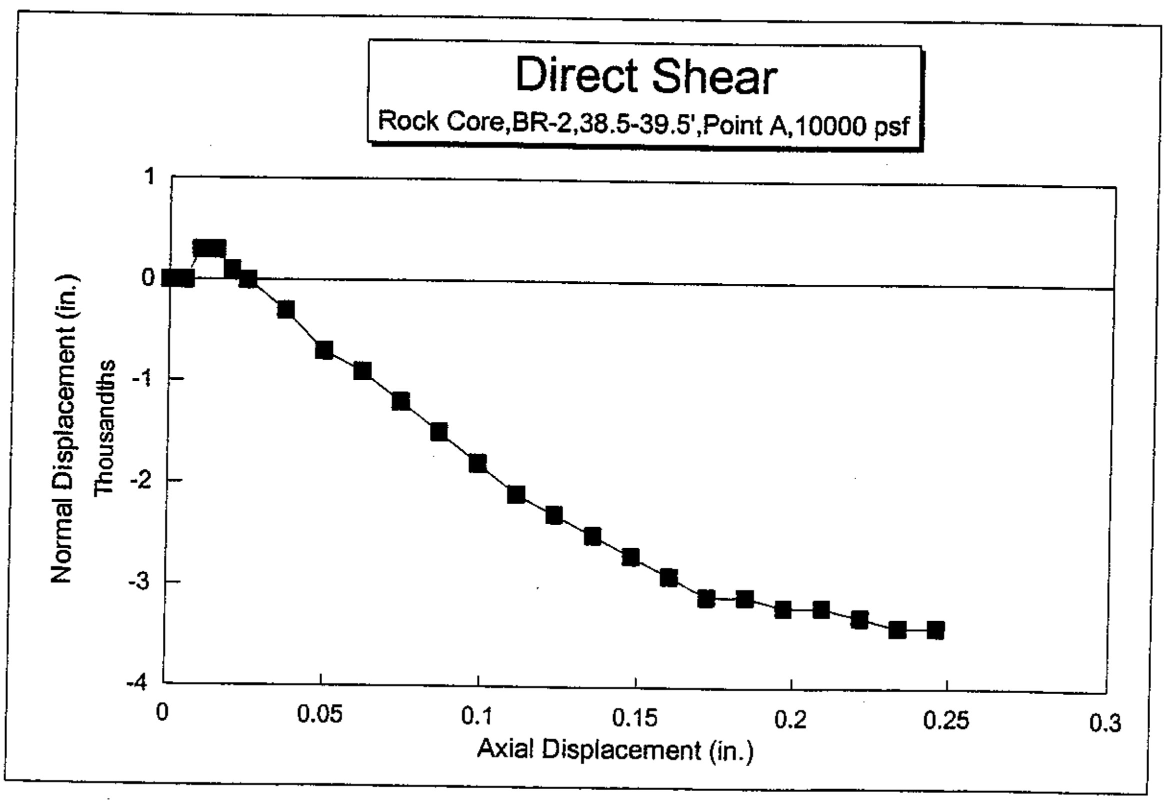
Client:	Goodson & Associates	Job Number:	2014-104
Boring:	BR-2	Date Tested:	06-27-06 DPM
Sample Number:	Rock Core	Soil Description:	
Depth:	38.5-39.5'	Point:	A
Location:	4th Street Bridge, SH 96	Normal Load	10000 PSF
		Peak Strength	4799 PSF

Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0049	82.0	2623	0.0000
0.0098	150.0	4799	0.0003
0.0148	150.0	4799	0.0003
0.0197	147.0	4703	0.0001
0.0246	144.0	4607	0.0000
0.0369	138.0	4415	-0.0003
0.0492	133.0	4255	-0.0007
0.0615	131.0	4191	-0.0009
0.0738	132.0	4223	-0.0012
0.0861	134.0	4287	-0.0015
0.0984	136.0	4351	-0.0018
0.1107	138.0	4415	-0.0021
0.1230	140.0	4479	-0.0023
0.1353	143.0	4575	-0.0025
0.1476	144.0	4607	-0.0027
0.1599	145.0	4639	-0.0029
0.1722	147.0	4703	-0.0031
0.1845	147.0	4703	-0.0031
0.1968	148.0	4735	-0.0032
0.2091	149.0	4767	-0.0032
0.2214	149.0	4767	-0.0033
0.2337	148.0	4735	-0.0034
0.2460	149.0	4767	-0.0034

Data entry by: SR Date: 06/27/2006
Checked by: RS Date: 7/27/06
FileName: GDDSBR2A

ADVANCED TERRA TESTING, INC


Direct Shear
Rock Core,BR-2,38.5-39.5',Point A,10000 psf
Shear Strength (PSF)
Thousands
0
1
2
3
4
5
6
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement


Direct Shear
Rock Core,BR-2,38.5-39.5',Point A,10000 psf
Normal Displacement (in.)
Thousandths
1
0
-1
-2
-3
-4
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement (in.)

DIRECT SHEAR TEST DATA
ASTM D3080

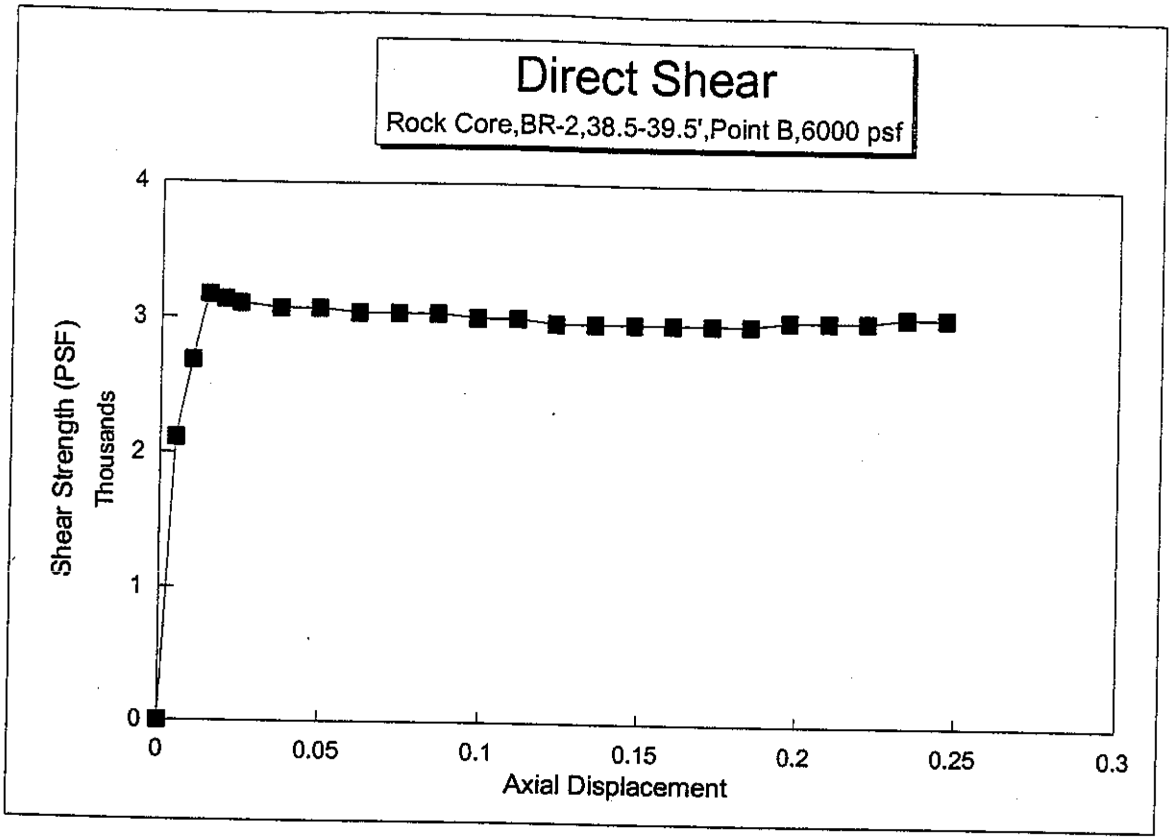
Client:	Goodson & Associates	Job Number:	2014-104
		Date Tested:	06-26-06 KB
Boring:	BR-2		
Sample Number:	Rock Core	Soil Description:	
Depth:	38.5-39.5'	Point:	B
Location:	4th Street Bridge, SH 96	Normal Load	6000 PSF
		Peak Strength	3167 PSF

Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0049	66.0	2111	-0.0001
0.0098	84.0	2687	0.0000
0.0148	99.0	3167	0.0001
0.0197	98.0	3135	0.0001
0.0246	97.0	3103	-0.0001
0.0369	96.0	3071	-0.0004
0.0492	96.0	3071	-0.0007
0.0615	95.0	3039	-0.0010
0.0738	95.0	3039	-0.0013
0.0861	95.0	3039	-0.0015
0.0984	94.0	3007	-0.0019
0.1107	94.0	3007	-0.0022
0.1230	93.0	2975	-0.0023
0.1353	93.0	2975	-0.0025
0.1476	93.0	2975	-0.0028
0.1599	93.0	2975	-0.0028
0.1722	93.0	2975	-0.0029
0.1845	93.0	2975	-0.0029
0.1968	94.0	3007	-0.0030
0.2091	94.0	3007	-0.0030
0.2214	94.0	3007	-0.0031
0.2337	95.0	3039	-0.0031
0.2460	95.0	3039	-0.0032

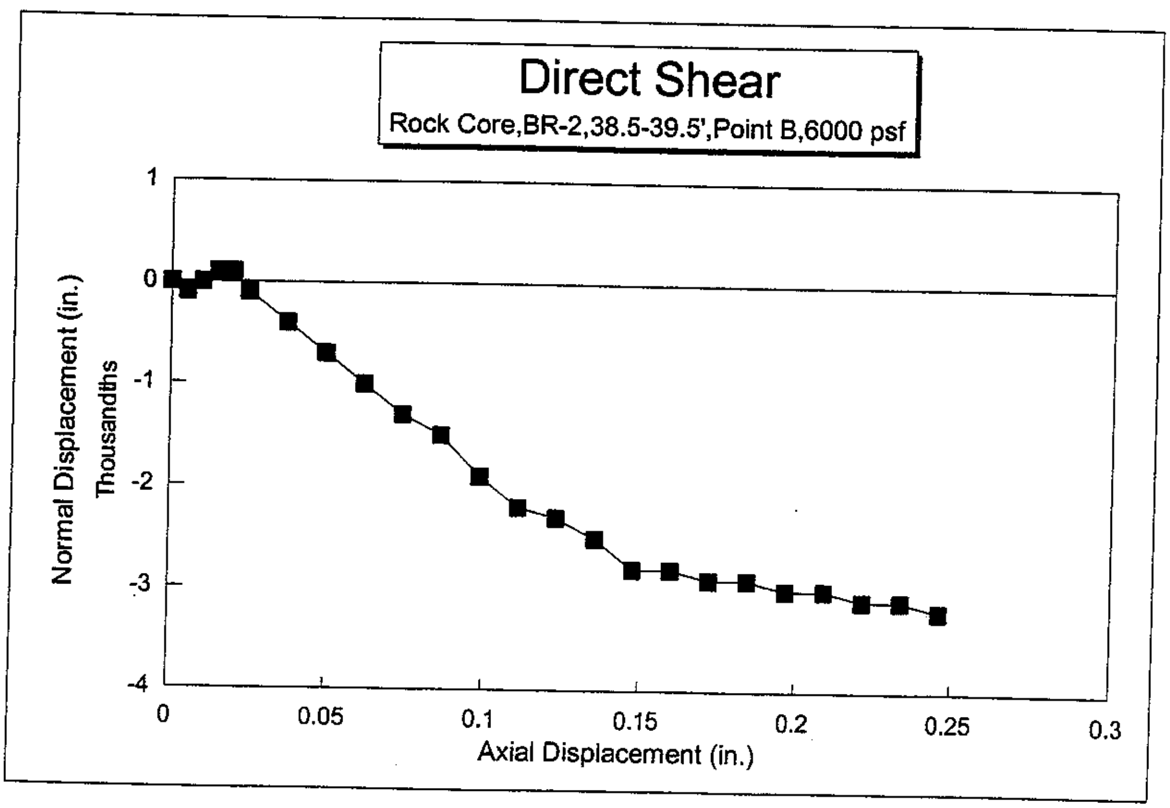
Data entry by: SR
Checked by: AL
FileName: GDDSBR2B

Date: 06/27/2006
Date: 7/17/06

ADVANCED TERRA TESTING, INC



Direct Shear
Rock Core,BR-2,38.5-39.5',Point B,6000 psf
Shear Strength (PSF)
Thousands
4
3
2
1
0
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement



Direct Shear
Rock Core,BR-2,38.5-39.5',Point B,6000 psf
Normal Displacement (in.)
Thousandths
1
0
-1
-2
-3
-4
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement (in.)

DIRECT SHEAR TEST DATA
ASTM D3080

Client: Goodson & Associates

Job Number: 2014-104
Date Tested: 06-23-06 DPM

Boring: BR-2
Sample Number: Rock Core
Depth: 38.5-39.5'
Location: 4th Street Bridge, SH 96

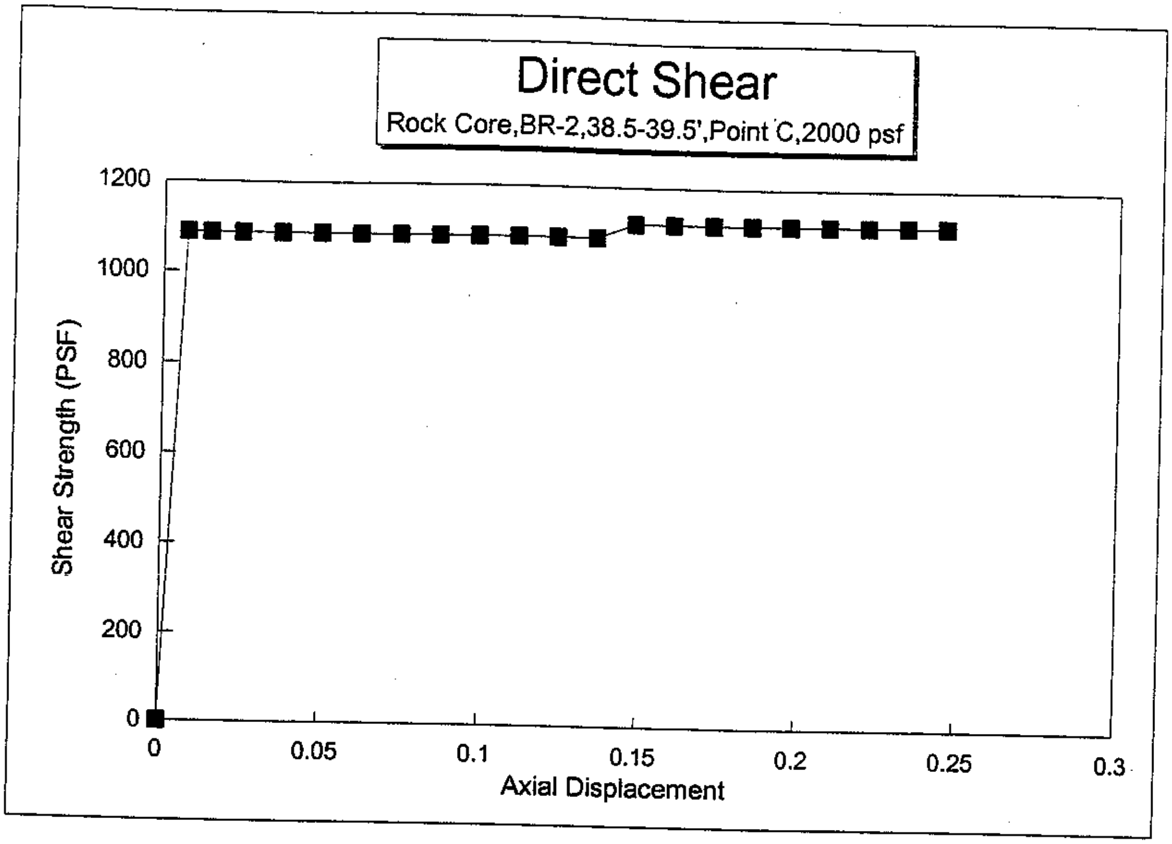
Soil Description:
Point: C
Normal Load 2000 PSF
Peak Strength 1120 PSF

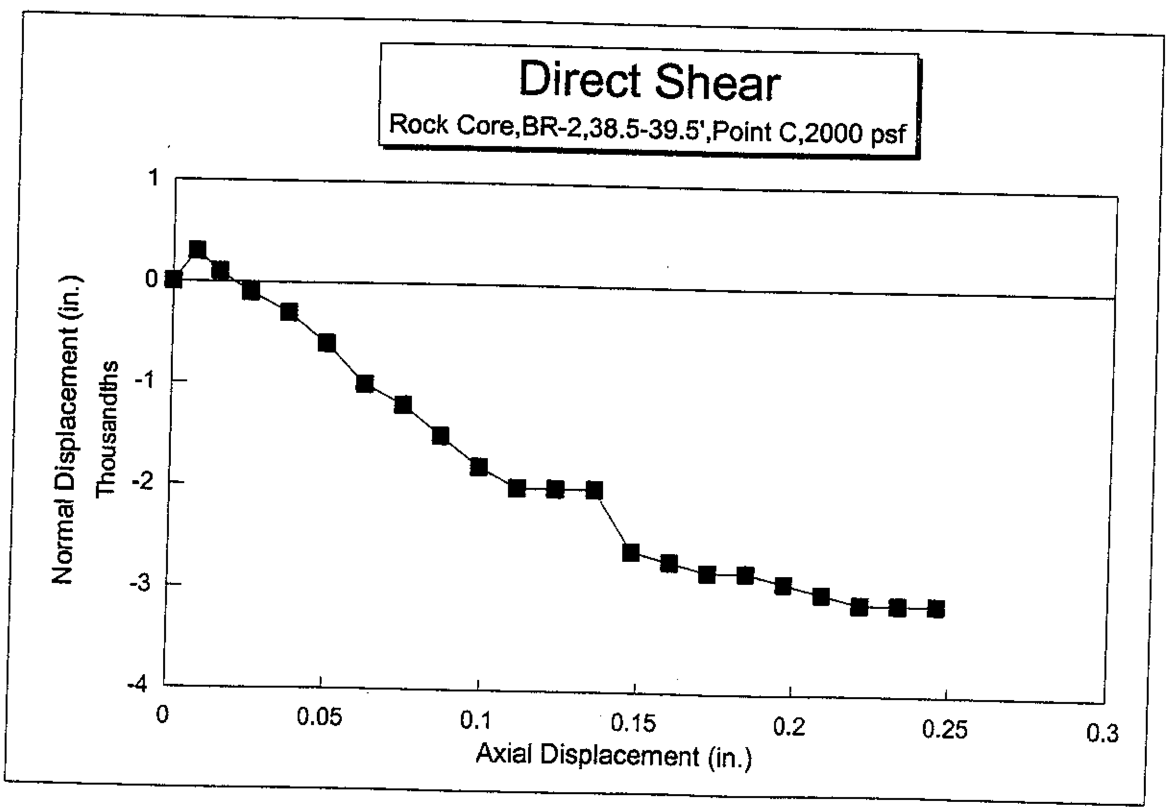
Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0074	34.0	1088	0.0003
0.0148	34.0	1088	0.0001
0.0246	34.0	1088	-0.0001
0.0369	34.0	1088	-0.0003
0.0492	34.0	1088	-0.0006
0.0615	34.0	1088	-0.0010
0.0738	34.0	1088	-0.0012
0.0861	34.0	1088	-0.0015
0.0984	34.0	1088	-0.0018
0.1107	34.0	1088	-0.0020
0.1230	34.0	1088	-0.0020
0.1353	34.0	1088	-0.0020
0.1476	35.0	1120	-0.0026
0.1599	35.0	1120	-0.0027
0.1722	35.0	1120	-0.0028
0.1845	35.0	1120	-0.0028
0.1968	35.0	1120	-0.0029
0.2091	35.0	1120	-0.0030
0.2214	35.0	1120	-0.0031
0.2337	35.0	1120	-0.0031
0.2460	35.0	1120	-0.0031
0.0000			
0.0000			

Data entry by: SR
Checked by: ND
FileName: GDDSBR2C

Date: 06/27/2006
Date: 7/17/06

ADVANCED TERRA TESTING, INC

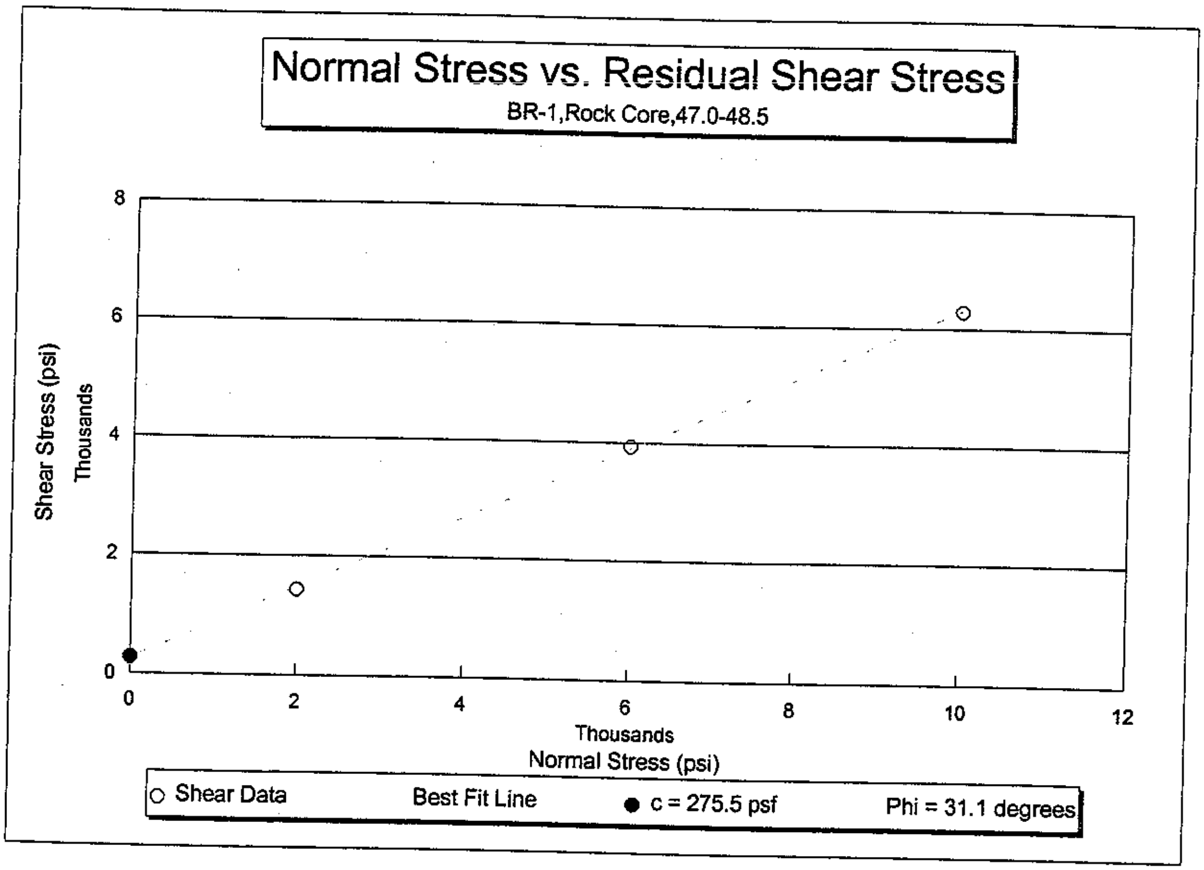

Direct Shear
Rock Core,BR-2,38.5-39.5',Point C,2000 psf
Shear Strength (PSF)
1200
1000
800
600
400
200
0
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement


Direct Shear
Rock Core,BR-2,38.5-39.5',Point C,2000 psf
Normal Displacement (in.)
Thousandths
1
0
-1
-2
-3
-4
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement (in.)

JOINT DIRECT SHEAR
ASTM D 3080 Modified

CLIENT	Goodson & Associates	JOB NO.	2014-104
BORING NO.	BR-1	DATE SAMPLED	
DEPTH	47.0-48.5	DATE TESTED	06-21-06 DPM
SAMPLE NO.	Rock Core	LOCATION	4th Street Bridge, SH 96
ROCK TYPE	Sandy Siltstone	JOINT TYPE	Saw Cut

NORMAL STRESS (psf)	RESIDUAL SHEAR STRESS (psf)
10000	6292
6000	3940
2000	1430



Notes and Comments:

Data entered by: SR Date: 06/26/2006
Data checked by: DPM Date: 6/27/06
File Name: GDDSBR1

ADVANCED TERRA TESTING, inc.

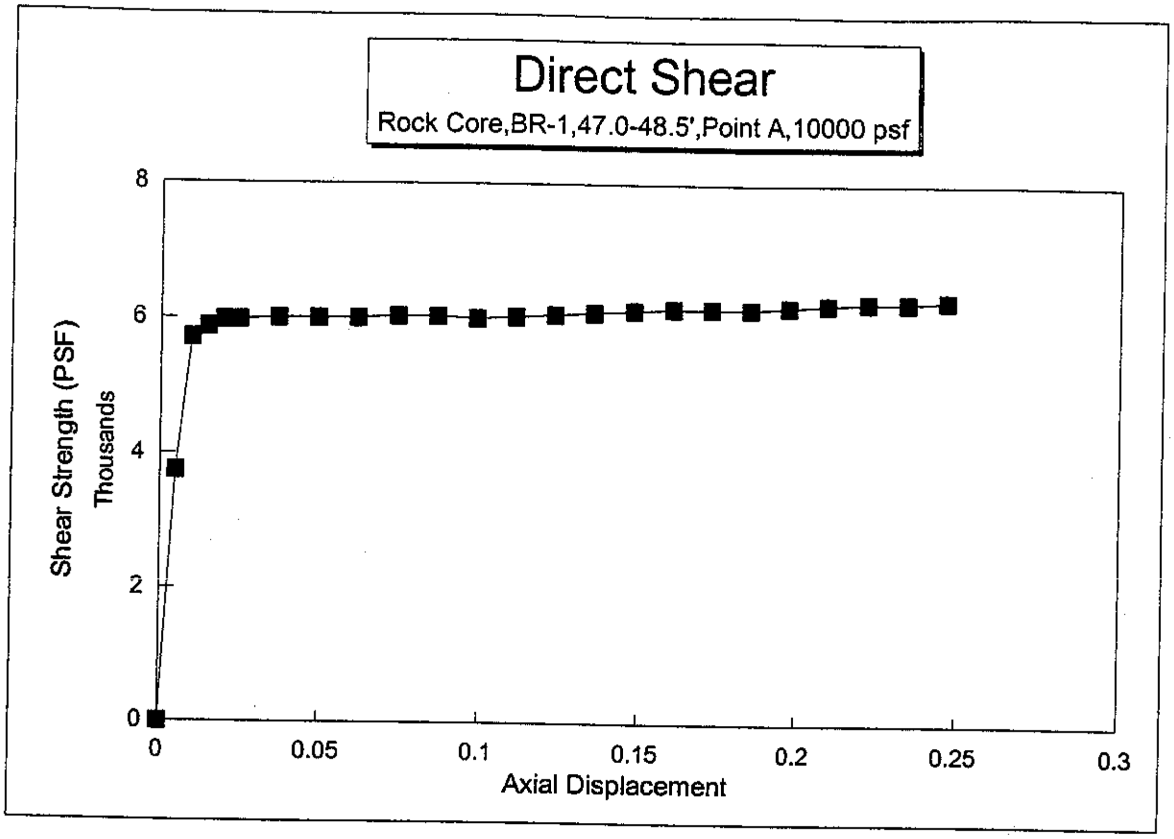
DIRECT SHEAR TEST DATA
ASTM D3080

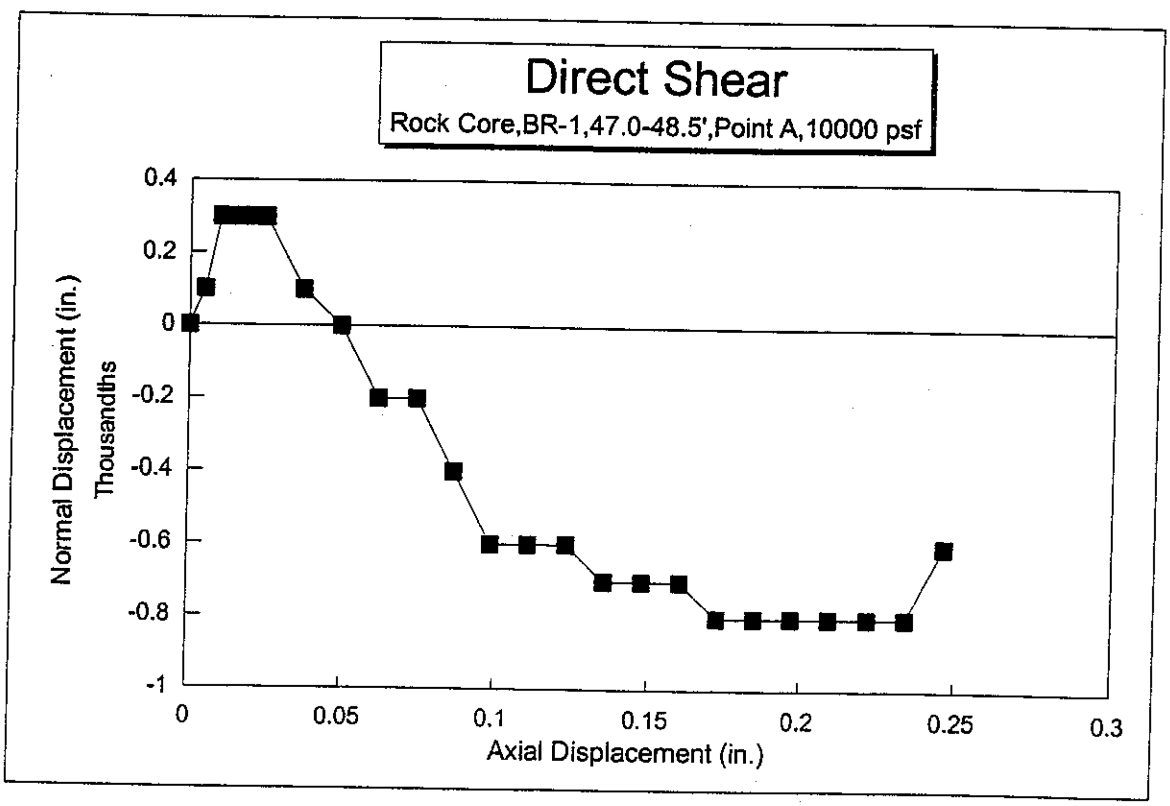
Client:	Goodson & Associates	Job Number:	2014-104
		Date Tested:	06-21-06 DPM
Boring:	BR-1		
Sample Number:	Rock Core	Soil Description:	
Depth:	47.0-48.5'	Point:	A
Location:	4th Street Bridge, SH 96	Normal Load	10000 PSF
		Peak Strength	6292 PSF

Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0049	118.0	3750	0.0001
0.0099	180.0	5720	0.0003
0.0148	185.0	5879	0.0003
0.0197	188.0	5974	0.0003
0.0247	188.0	5974	0.0003
0.0370	189.0	6006	0.0001
0.0493	189.0	6006	0.0000
0.0616	189.0	6006	-0.0002
0.0740	190.0	6038	-0.0002
0.0863	190.0	6038	-0.0004
0.0986	189.0	6006	-0.0006
0.1109	190.0	6038	-0.0006
0.1233	191.0	6070	-0.0006
0.1356	192.0	6101	-0.0007
0.1479	193.0	6133	-0.0007
0.1602	194.0	6165	-0.0007
0.1726	194.0	6165	-0.0008
0.1849	194.0	6165	-0.0008
0.1972	195.0	6197	-0.0008
0.2095	196.0	6228	-0.0008
0.2219	197.0	6260	-0.0008
0.2342	197.0	6260	-0.0008
0.2465	198.0	6292	-0.0006

Data entry by: SR   Date: 06/22/2006
Checked by: DPM   Date: 6/26/06
FileName: GDDSBR1A

ADVANCED TERRA TESTING, INC


Direct Shear
Rock Core,BR-1,47.0-48.5',Point A,10000 psf
Shear Strength (PSF)
Thousands
8
6
4
2
0
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement


Direct Shear
Rock Core,BR-1,47.0-48.5',Point A,10000 psf
Normal Displacement (in.)
Thousandths
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement (in.)

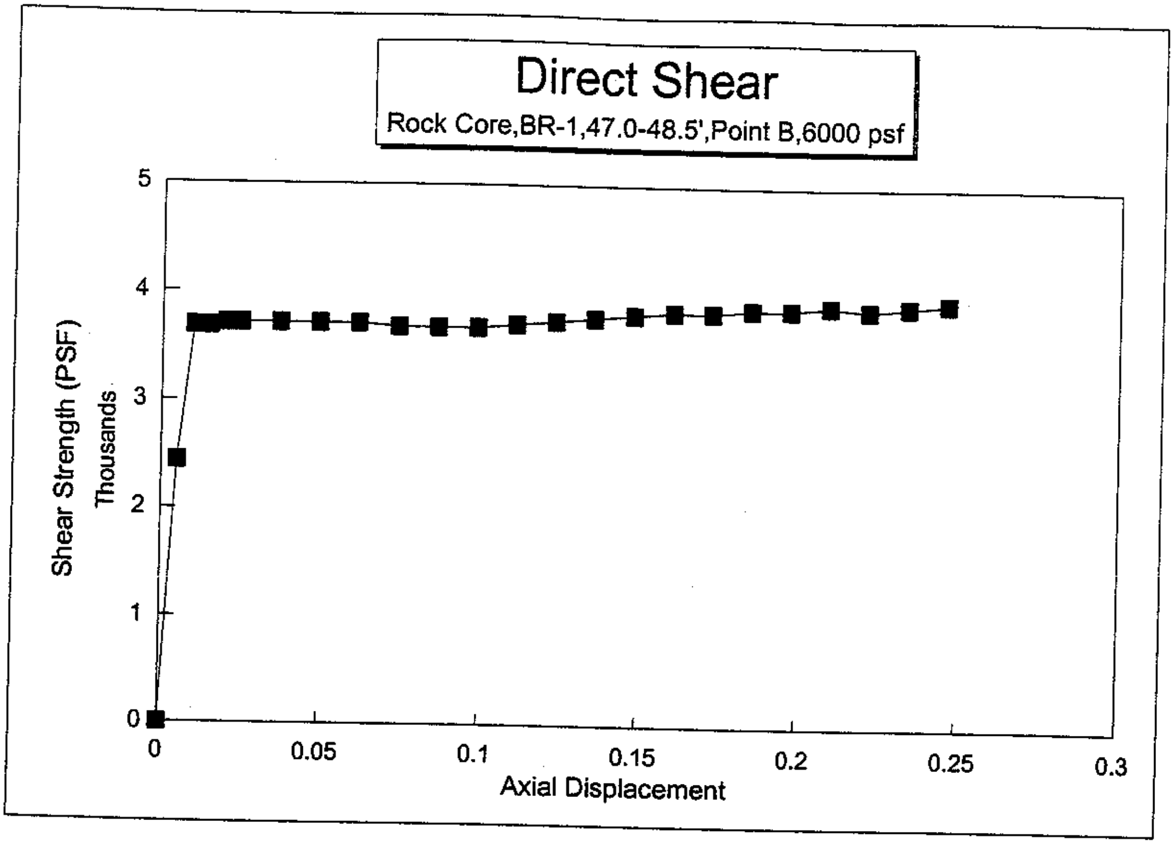
DIRECT SHEAR TEST DATA
ASTM D3080

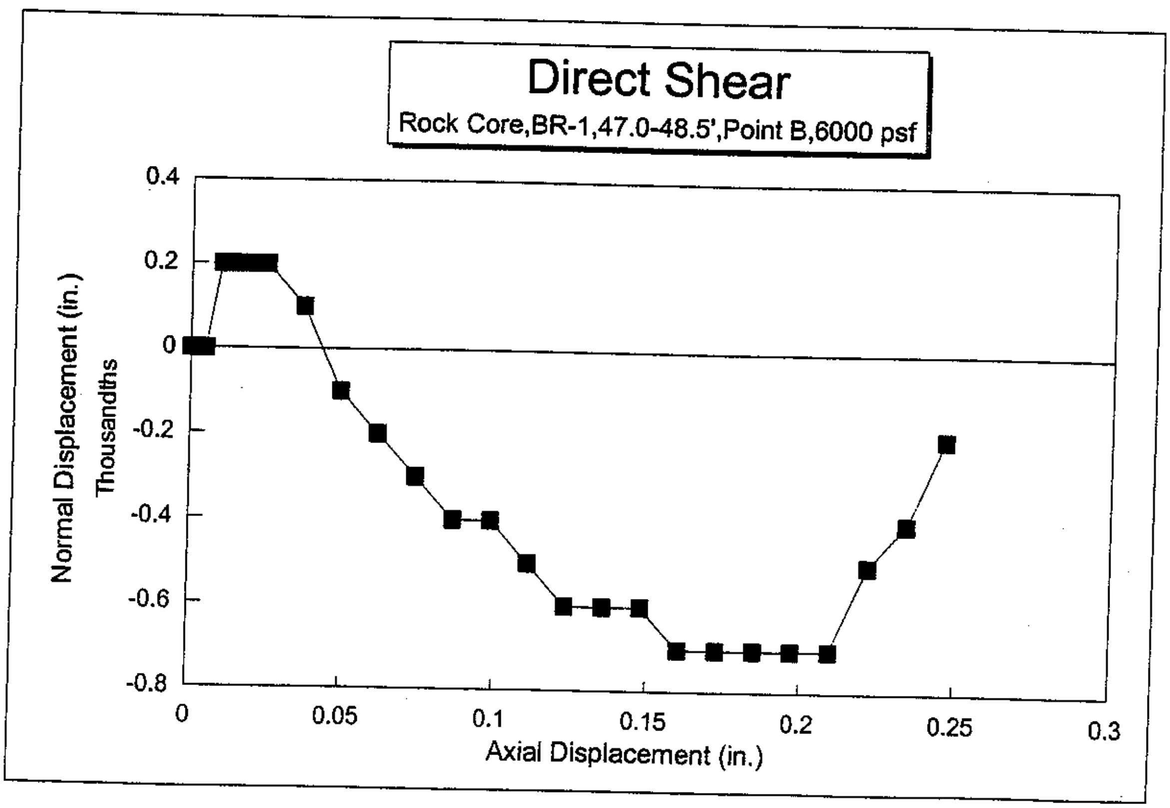
Client:	Goodson & Associates	Job Number:	2014-104
		Date Tested:	06-21-06 DPM
Boring:	BR-1		
Sample Number:	Rock Core	Soil Description:	
Depth:	47.0-48.5'	Point:	B
Location:	4th Street Bridge, SH 96	Normal Load	6000 PSF
		Peak Strength	3940 PSF

Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0049	77.0	2447	0.0000
0.0099	116.0	3686	0.0002
0.0148	116.0	3686	0.0002
0.0197	117.0	3718	0.0002
0.0247	117.0	3718	0.0002
0.0370	117.0	3718	0.0001
0.0493	117.0	3718	-0.0001
0.0616	117.0	3718	-0.0002
0.0740	116.0	3686	-0.0003
0.0863	116.0	3686	-0.0004
0.0986	116.0	3686	-0.0004
0.1109	117.0	3718	-0.0005
0.1233	118.0	3750	-0.0006
0.1356	119.0	3782	-0.0006
0.1479	120.0	3813	-0.0006
0.1602	121.0	3845	-0.0007
0.1726	121.0	3845	-0.0007
0.1849	122.0	3877	-0.0007
0.1972	122.0	3877	-0.0007
0.2095	123.0	3909	-0.0007
0.2219	122.0	3877	-0.0005
0.2342	123.0	3909	-0.0004
0.2465	124.0	3940	-0.0002

Data entry by: SR Date: 06/22/2006
Checked by: DPM Date: 6/26/06
FileName: GDDSBR1B

ADVANCED TERRA TESTING, INC


Direct Shear
Rock Core,BR-1,47.0-48.5',Point B,6000 psf
Shear Strength (PSF)
Thousands
Axial Displacement
0
1
2
3
4
5
0.05
0.1
0.15
0.2
0.25
0.3


Direct Shear
Rock Core,BR-1,47.0-48.5',Point B,6000 psf
Normal Displacement (in.)
Thousandths
Axial Displacement (in.)
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
0.05
0.1
0.15
0.2
0.25
0.3

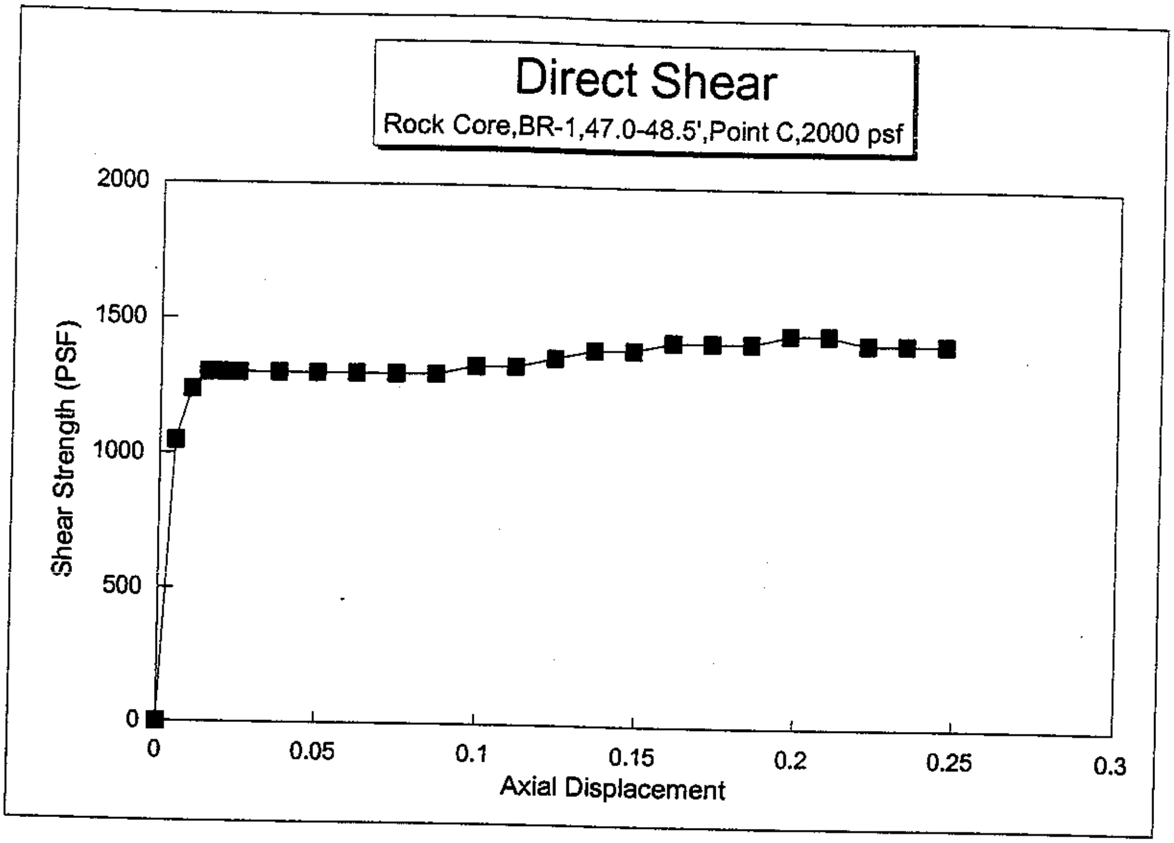
DIRECT SHEAR TEST DATA
ASTM D3080

Client:	Goodson & Associates	Job Number:	2014-104
		Date Tested:	06-21-06 DPM/KB
Boring:	BR-1		
Sample Number:	Rock Core	Soil Description:	
Depth:	47.0-48.5'	Point:	C
Location:	4th Street Bridge, SH 96	Normal Load	2000 PSF
		Peak Strength	1462 PSF

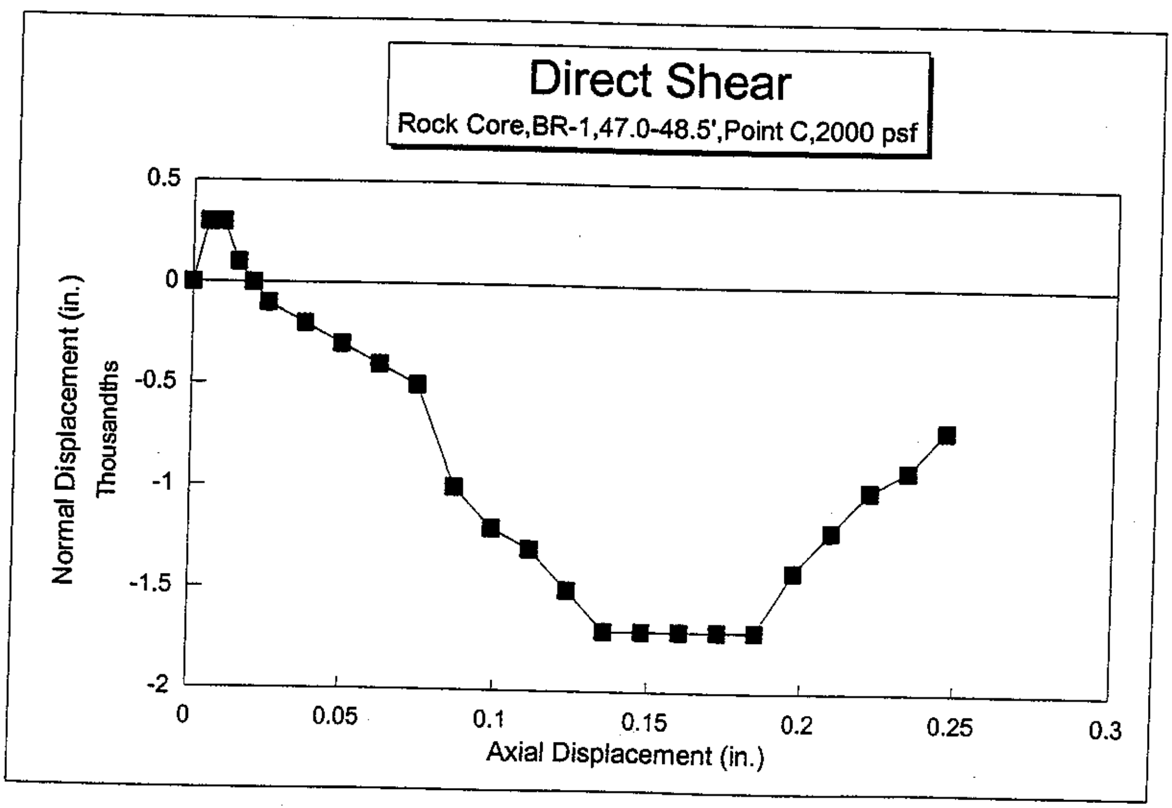
Shear Displacement (inches)	Shear Load (lbs.)	Shear Load (PSF)	Normal Displacement (inches)
0.0000	0.0	0	0.0000
0.0049	33.0	1049	0.0003
0.0099	39.0	1239	0.0003
0.0148	41.0	1303	0.0001
0.0197	41.0	1303	0.0000
0.0247	41.0	1303	-0.0001
0.0370	41.0	1303	-0.0002
0.0493	41.0	1303	-0.0003
0.0616	41.0	1303	-0.0004
0.0740	41.0	1303	-0.0005
0.0863	41.0	1303	-0.0010
0.0986	42.0	1335	-0.0012
0.1109	42.0	1335	-0.0013
0.1233	43.0	1366	-0.0015
0.1356	44.0	1398	-0.0017
0.1479	44.0	1398	-0.0017
0.1602	45.0	1430	-0.0017
0.1726	45.0	1430	-0.0017
0.1849	45.0	1430	-0.0017
0.1972	46.0	1462	-0.0014
0.2095	46.0	1462	-0.0012
0.2219	45.0	1430	-0.0010
0.2342	45.0	1430	-0.0009
0.2465	45.0	1430	-0.0007

Data entry by: SR Date: 06/22/2006
Checked by: DPM Date: 6/26/06
FileName: GDDSBR1C

ADVANCED TERRA TESTING, INC



Direct Shear
Rock Core,BR-1,47.0-48.5',Point C,2000 psf
Shear Strength (PSF)
2000
1500
1000
500
0
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement



Direct Shear
Rock Core,BR-1,47.0-48.5',Point C,2000 psf
Normal Displacement (in.)
Thousandths
0.5
0
-0.5
-1
-1.5
-2
0
0.05
0.1
0.15
0.2
0.25
0.3
Axial Displacement (in.)

UNCONFINED COMPRESSIVE STRENGTH
WITH STRESS/STRAIN MEASUREMENTS
ASTM D 3148

UNCONFINED COMPRESSIVE STRENGTH
With Stress / Strain Measurements
ASTM D 3148

CLIENT: Goodson & associates, Inc.

LOCATION: 4th Street Bridge

JOB NO.: 2014-104

DATE TESTED: 7/3/06 HN

Specimen ID			Diameter (in.)	Length (in.)	Mass (gms)	Wet Density (pcf)	Failure Load (lb)	Failure Type **	Compressive Strength (psi)	Young's Modulus (X10^6 psi)	Poisson's Ratio
Boring	Depth (ft.)	Rock type									
BR-1	41.0-42.0		2.407	5.228	947.20	151.7	40,250	S/F	8,850	1.21	0.083
BR-1	47.0-48.5		2.410	5.348	942.10	147.1	11,250	S	2,470	0.99	0.064
BR-2	30.2-32.0		2.403	5.247	918.30	147.0	49,250	C	10,860	1.38	0.122
BR-2	45.7-47.2		2.408	5.402	939.90	145.5	7,625	S	1,670	0.29	0.041
BR-3	11.1-11.8		2.401	3.943	657.40	140.3	5,800	S	*1250	0.12	0.041
BR-3	24.0-25.0		2.405	4.963	890.10	150.4	4,750	F	1,050	0.15	0.024

Notes and Comments:

* Indicates L/D < 2.0. Correction Factor for short sample was applied..

C=Ca/[0.88+0.24b/h]

Ca = Failure Load / Surface Area

b = Sample Diameter

h = Sample Length

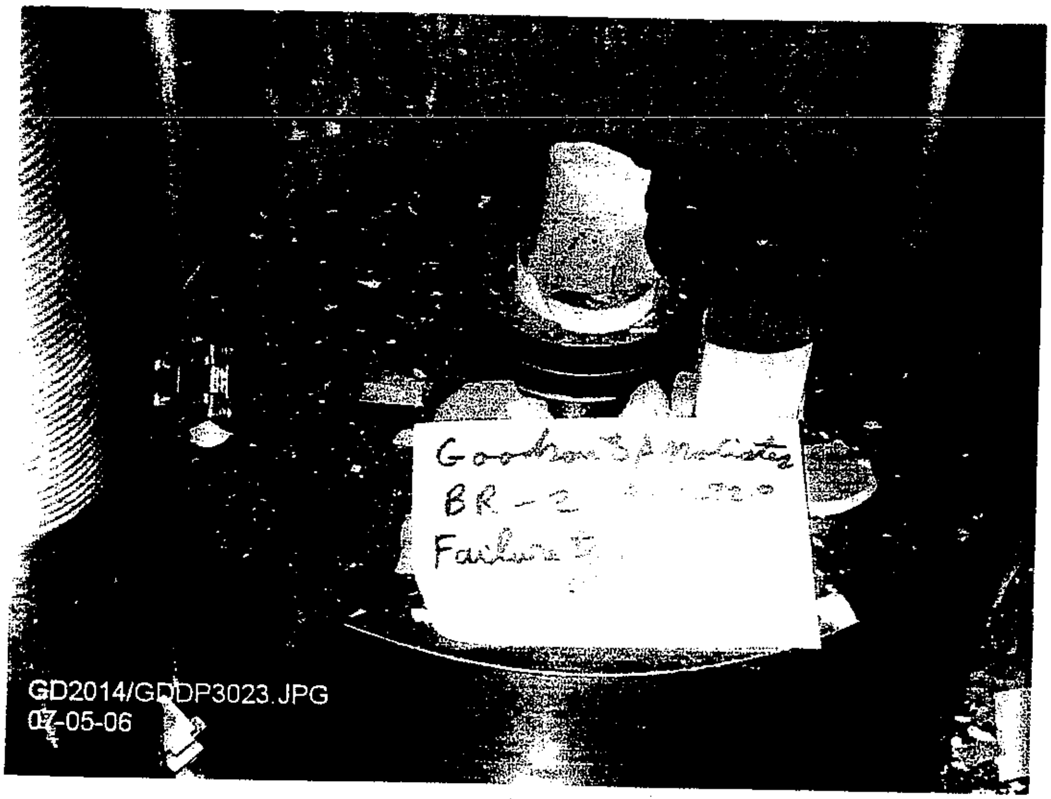
** Failure Type:

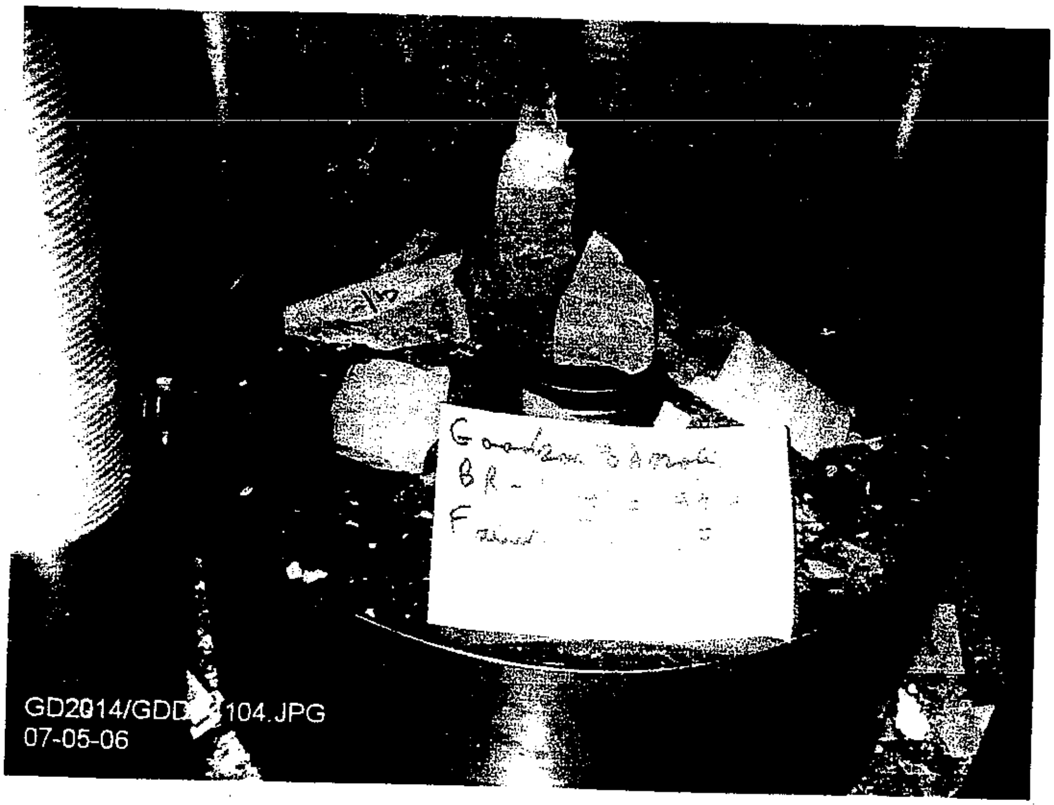
S: Shear Failure, M: Matrix Failure, F/V Fracture, Bedding/Void Collapse, C: Combination

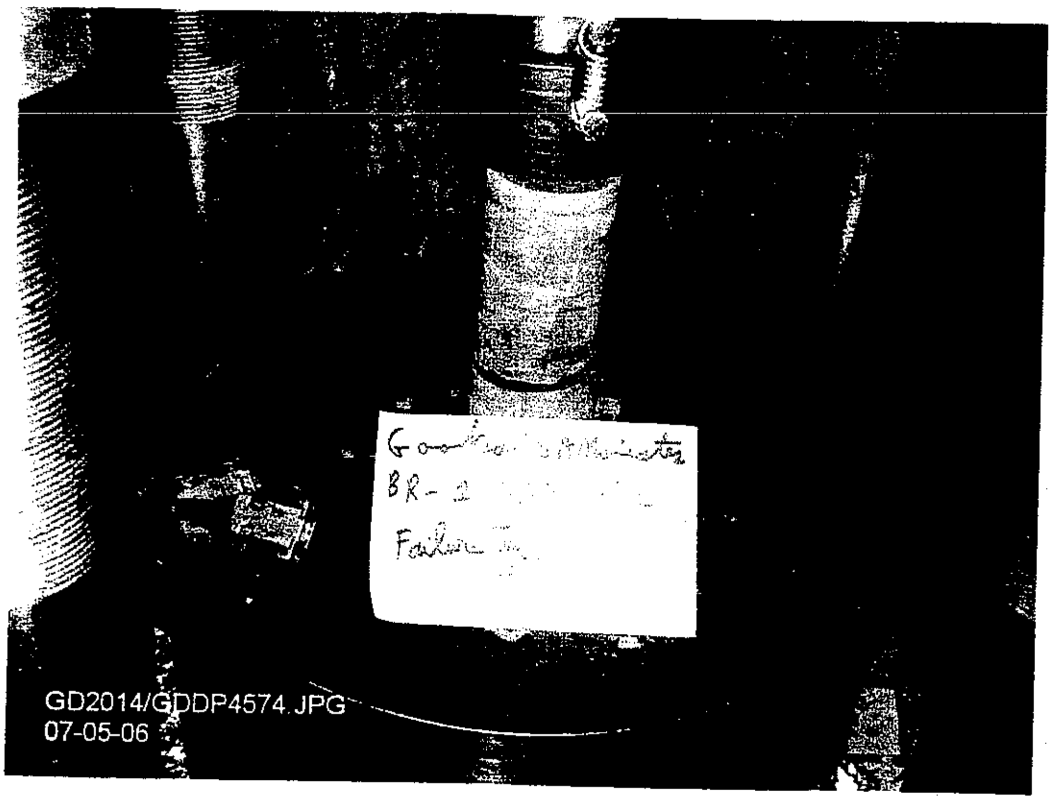
Data Entered By: HN Date: 07/03/2006
Data Checked By: CJ Date: 07 05 06
Filename: GDUCSSR1

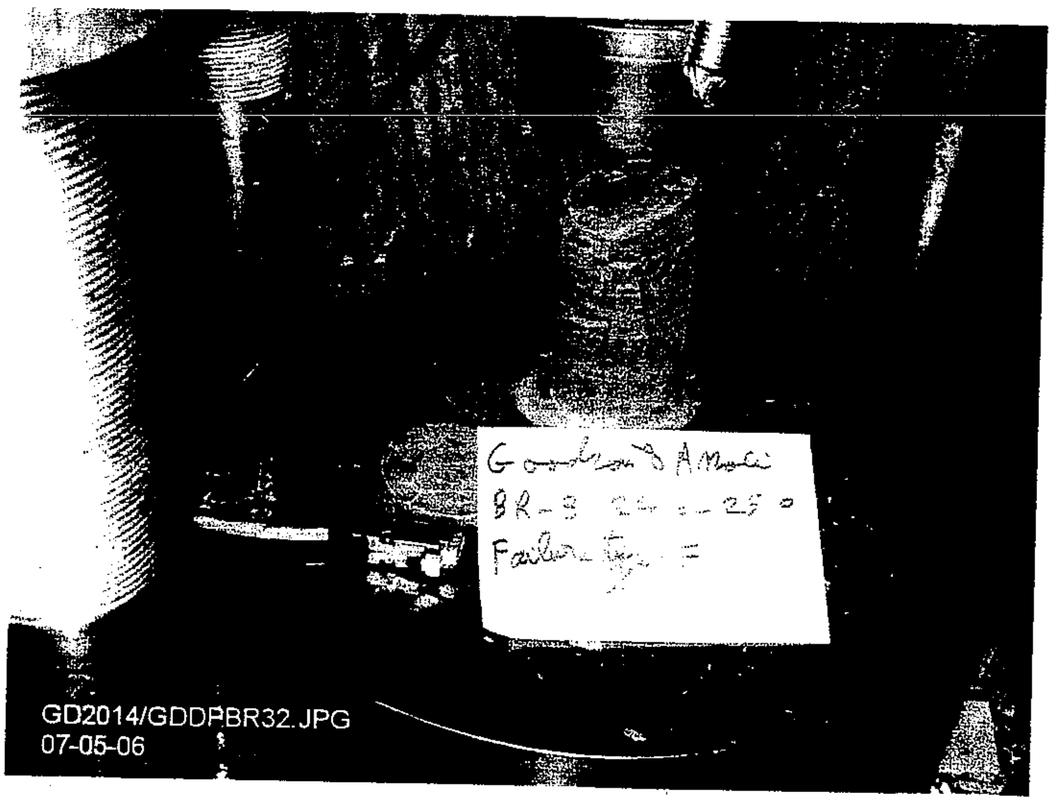
ADVANCED TERRA TESTING, Inc.

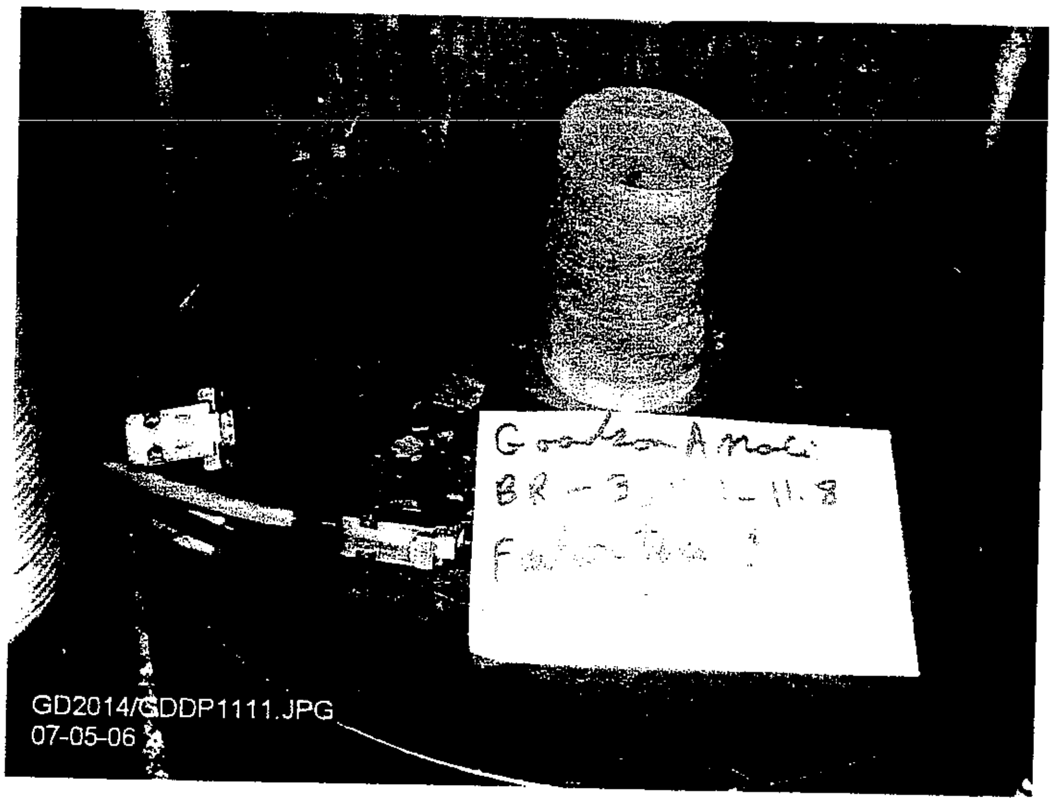

BR-1, 47.0 - 48.5
Failure
GD2014/GDDP4704.JPG
07-05-06

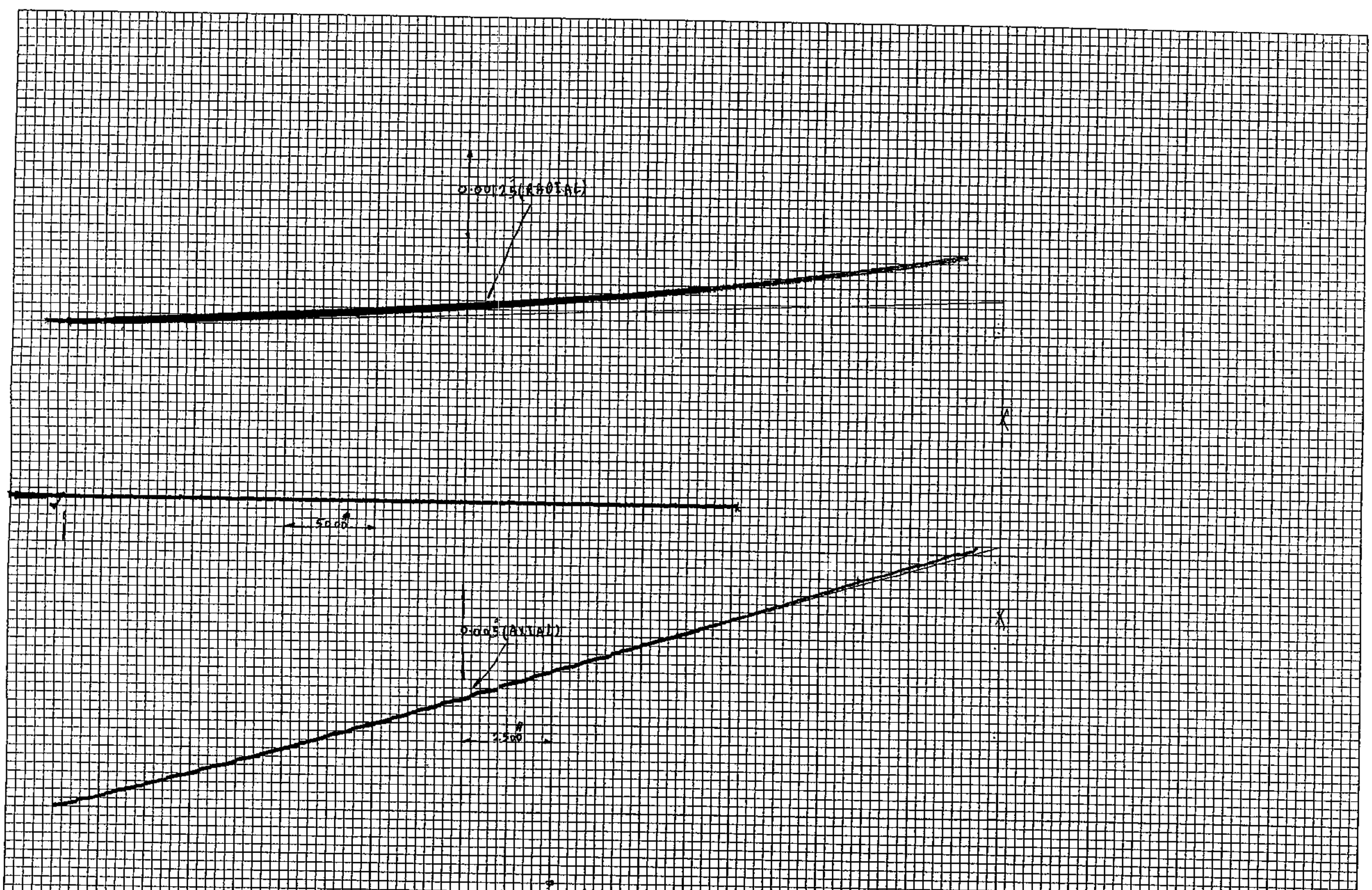

BR - 2
Failure
GD2014/GDDP3023.JPG
07-05-06


GD2014/GDD 104.JPG
07-05-06


BR-2
GD2014/GDDP4574.JPG
07-05-06


GD2014/GDDPBR32.JPG
07-05-06


GD2014/GDDP1111.JPG
07-05-06



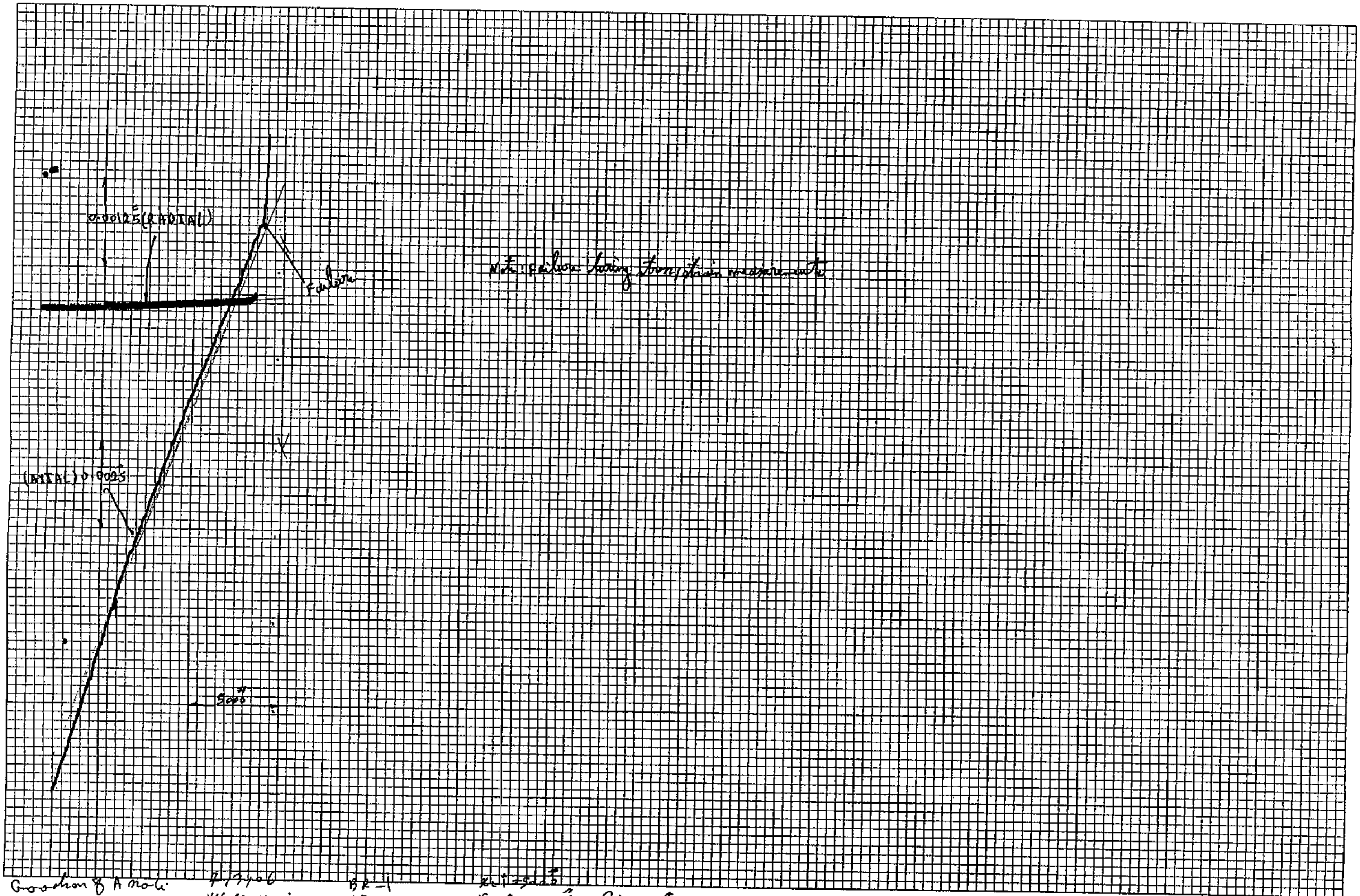
Goodman & Amadi
#2014-104

7/3/06 HR
50 Kg UCS/SS

BR-1
41-42

x1 = 2500 lbs
Y1 L = .005
Y2 R = .00125

GL = 3.25



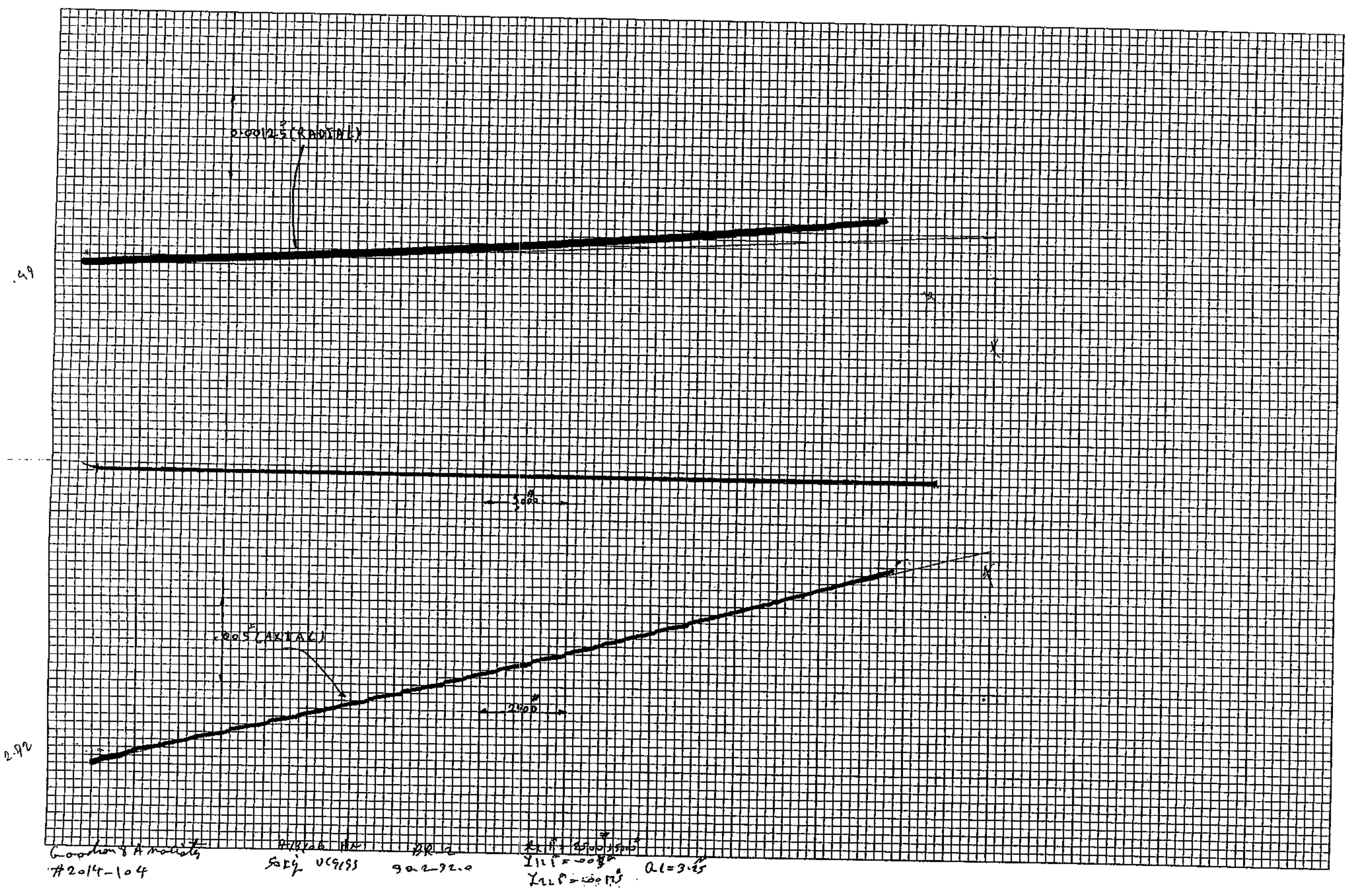
Goodman & A Mole
2014-104

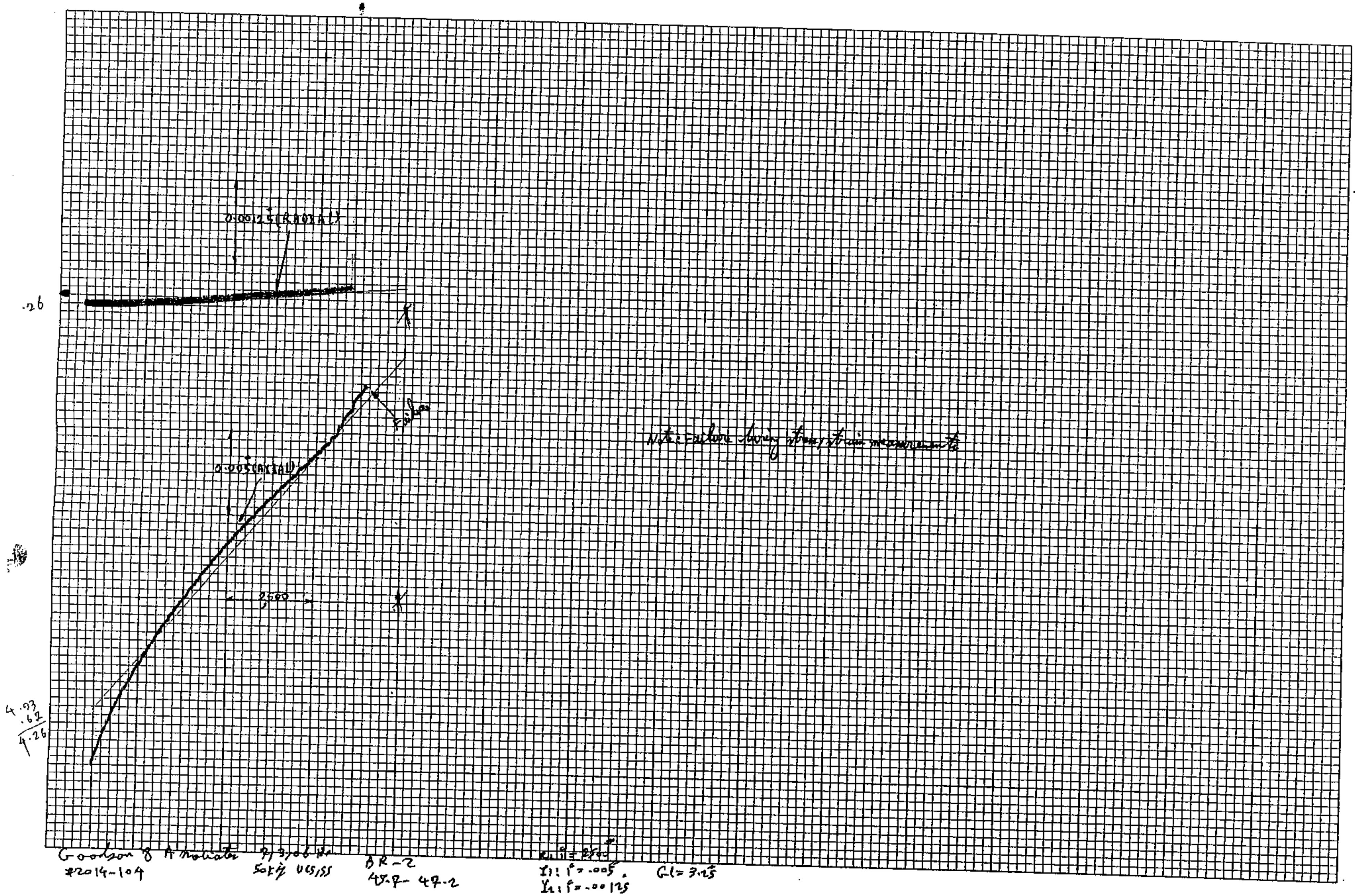
7/7/06
V19155 50kg

BR-1
47. -48.5

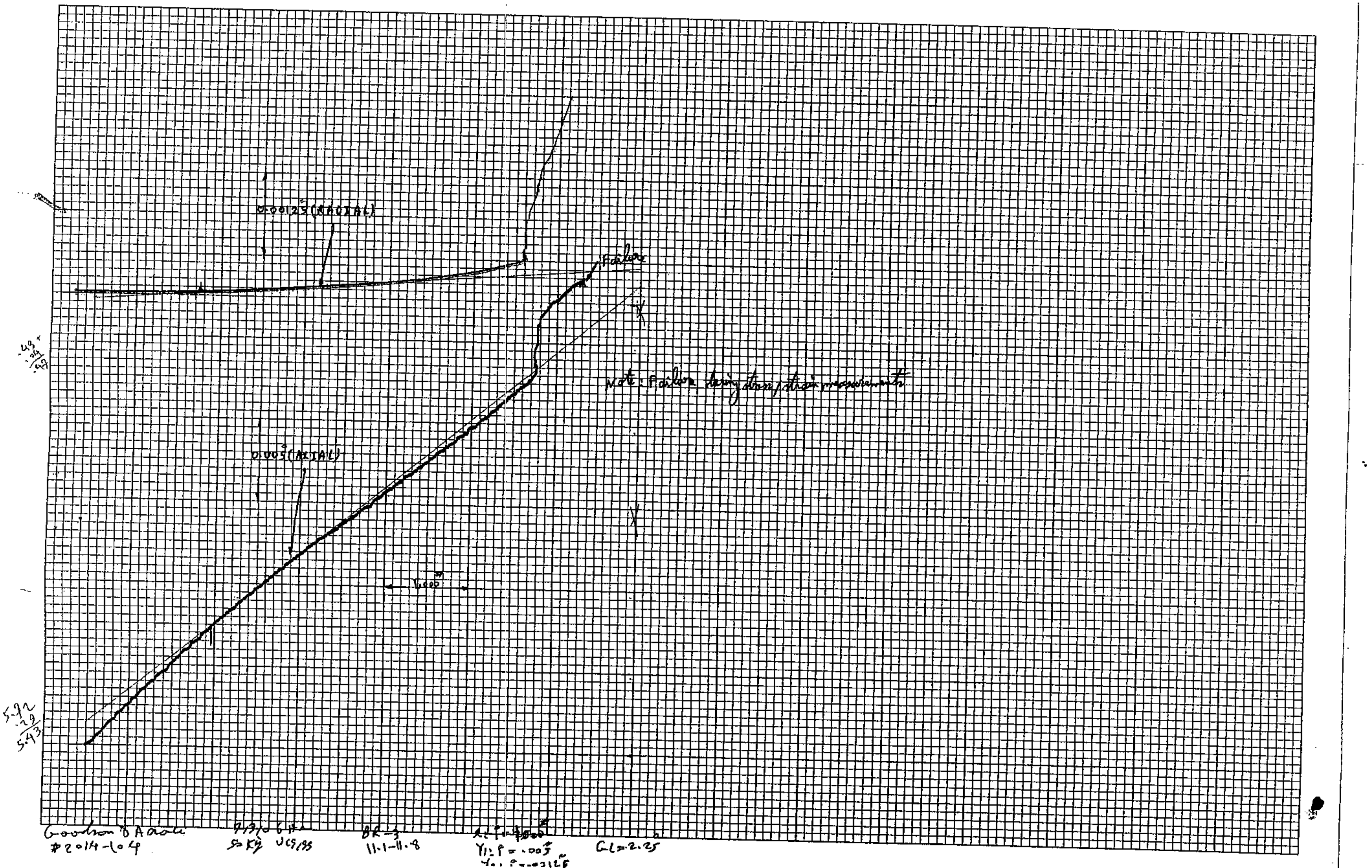
X: 1 = 5000#
C: 1 = .0025
r: 1 = .00125

GL = 3.25

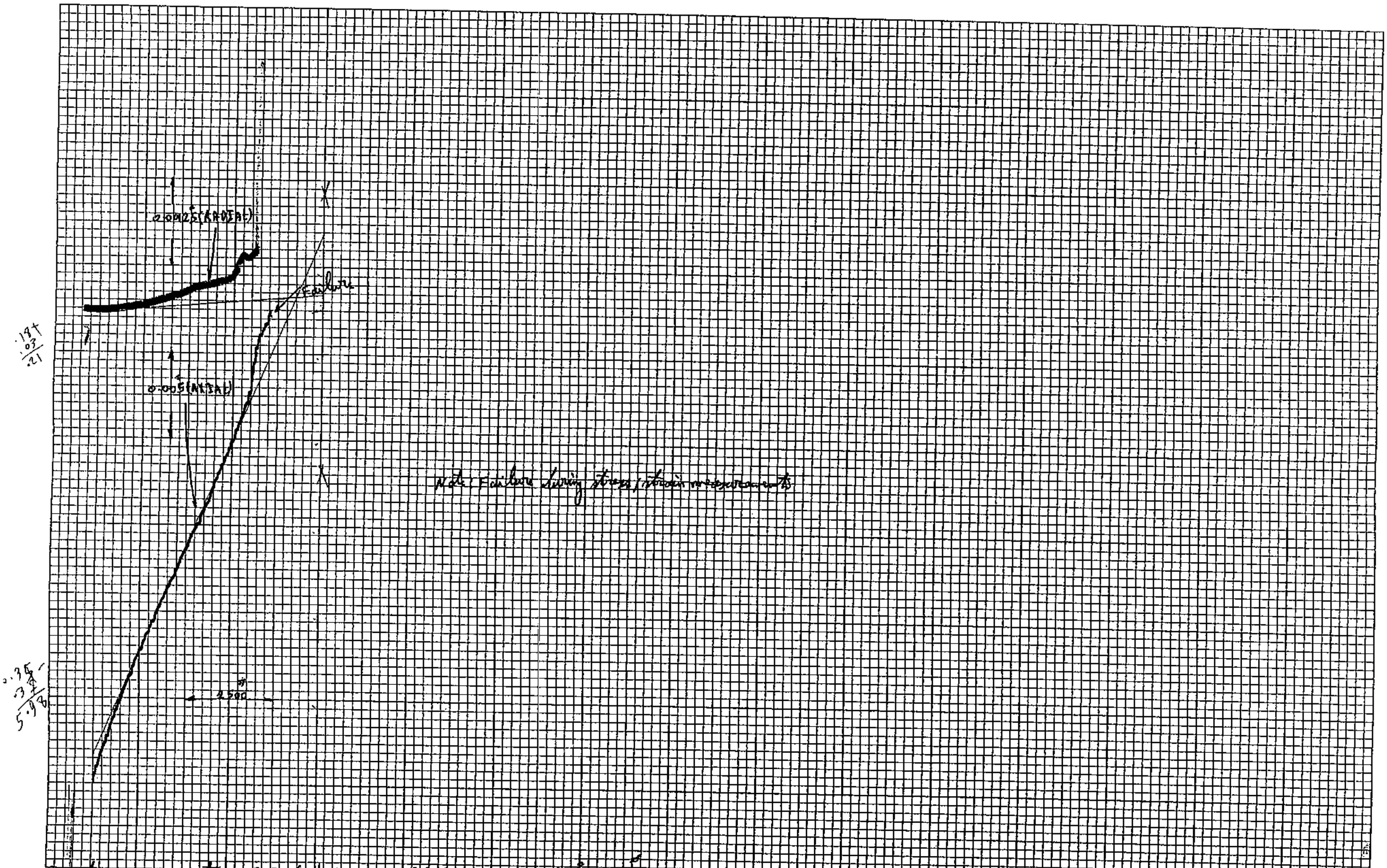

0.00125" (RADIAL)
.49
5000#
.005" (AXIAL)
2500#
2.92
#2014-104
BR-2
A1=3.25"



.26
0.00125" (RADIAL)
0.005" (AXIAL)
2500
Failure
4.93
.67
4.26
Note: Failure during stress/strain measurements
#2014-104
BR-2
G.L = 3.25"



0.00125" (RADIAL)
Failure
0.005" (AXIAL)
1,000#
Note: Failure during stress/strain measurements
5.92
.49
5.43
#2014-104
BR-3
11.1-11.8
GL=2.25



Gordon & Amaliate
#2014-104

9/3/06
50 Kg UC9455

BR-3
24-25

X1 = 0.005
Y2 = 0.00125

GL = 3.25