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Supplemental Geotechnical Investigation
Pine and Tuscarora Remediation Site
Niagara Falls, New York

Woodward-Clyde Consultants

Consulting Engineers, Geologists and Environmental Scientists

5120 Butler Pike, Plymouth Meeting, Pennsylvania 19462

5120 Butler Pike
Plymouth Meeting
Pennsylvania 19462
215-825-3000
Fax 215-834-0234

Woodward-Clyde Consultants

February 23, 1990
89C285 7

Olin Chemicals
Lower River Road
P.O. Box 248
Charleston, TN 37310

Attention: Mr. James D. Frye

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PINE AND TUSCARORA REMEDIATION SITE
NIAGARA FALLS, NEW YORK

Gentlemen:

Woodward-Clyde Consultants (WCC) is pleased to present this Supplemental Geotechnical Investigation for the Pine and Tuscarora Remediation Site. This work was performed in accordance with our proposal dated December 21, 1989, and Olin's subsequent authorization.

Soil samples obtained during this investigation will be retained in our laboratory for a period of three months unless other instructions are received regarding their final disposition.

Woodward-Clyde Consultants appreciates the opportunity of providing these services to Olin Chemicals. Should you have any questions, or if we could be of further service, please do not hesitate to contact us.

Very truly yours,

WOODWARD-CLYDE CONSULTANTS

Frank S. Waller

Frank S. Waller, P.E.
Senior Managing Principal

FSW/mek

SUPPLEMENTAL GEOTECHNICAL INVESTIGATION
PINE AND TUSCARORA REMEDIATION SITE
NIAGARA FALLS, NEW YORK

Prepared for:

OLIN CHEMICALS

Charleston, Tennessee

Prepared by:

WOODWARD-CLYDE CONSULTANTS

Plymouth Meeting, Pennsylvania

TABLE OF CONTENTS

LIST OF FIGURES

APPENDICES

INTRODUCTION

This report presents the results of a Supplemental Geotechnical Investigation performed at the Olin Pine and Tuscarora (Gibson) Remediation Site located in Niagara Falls, New York, as shown on Figure 1. This supplemental investigation was necessary to answer technical questions raised by the New York State Department of Environmental Conservation (NYSDEC) in project review correspondences and meetings. The work was performed in accordance with Woodward-Clyde Consultants' (WCC) proposal dated December 21, 1989, and Olin's subsequent authorization.

The primary purposes of the investigation were as follows:

- To secure representative undisturbed soil samples for laboratory engineering property testing
- To further delineate the subsurface conditions in the area of the proposed channel relocation and filling
- To perform engineering analyses relative to embankment stability and settlement, slope protection, and slurry wall design and construction
- To modify the existing design, if required, based upon the results of this supplemental investigation

The scope of work was limited to performing three additional test borings (P-5, WCC-5, and WCC-6), conducting various laboratory tests on representative samples, performing appropriate engineering analyses, and the preparation of this report.

FIELD INVESTIGATION

The field investigation consisted of performing three test borings at the locations shown on Figure 2 (Survey Drawing by Wendel Engineers). Wendel Engineers of Lockport, New York performed the boring layout and elevation survey. The test borings were drilled by Empire Soils of Hamburg, New York. Borings P-5 and WCC-5 were drilled in the low-lying floodplain area north of the Pine and Tuscarora Remediation Site. Boring P-5 was drilled immediately adjacent to the existing Cayuga Creek channel so that disturbed soil samples could be obtained of the soils beneath the existing channel. Boring WCC-5 was drilled approximately 65 feet north of Boring P-5, so that undisturbed samples could be obtained from the area of the proposed edge of disposal area cap, slurry wall, and embankment slope. These two borings are located in an area that is believed to represent the most critical construction area for the proposed embankment and slurry wall. Boring WCC-6 was performed along the east side (see Figure 2) of the remediation site, on the higher and more stable portion of the proposed construction area.

A description of the field investigation program is included in Appendix A, together with the logs of the test borings and a key to the symbols and terms used in this report. Results of other field exploration data were used, but for brevity, these are not repeated herein, as they have been previously included with other technical reports.

LABORATORY INVESTIGATION

The laboratory investigation consisted of physical and engineering property tests conducted on representative samples obtained from the three additional test borings. Of primary concern were the engineering properties (strength, compressibility, and permeability) of the very soft clay layer present between approximate elevations 561 and 553, although some tests were performed on samples above and below these elevations for soil profile characterization purposes. A description of the laboratory testing program is presented in Appendix B, together with the test results. The test results are also summarized in the Subsurface Conditions section of this report.

SITE CONDITIONS

Cayuga Creek flows in a general southerly direction at the northern and eastern edges of the Pine and Tuscarora Remediation Site, as shown on Figure 2. The existing channel bottom ranges from about elevation 560 to 561 as it passes the remediation site. Generally, the channel bottom is about 10 to 15 feet wide, but in some locations the channel meander has cut a wider channel (up to about 40 feet). The normal flow is only about 8 to 10 cfs and over 25 percent of the time the flow is less than 3 cfs. However, design storm flows are reported[1] to be very substantial due to the 14.3 square mile drainage area as listed below:

Return Period (years)	Peak Flow Volume (cfs)
10	950
100	1650
500	2100

The 100-year flood level is approximately elevation 571 (10-foot depth of water) adjacent to the remediation site. The flood of record has an elevation of 567.7 adjacent to the site.

SUBSURFACE CONDITIONS

The subsurface conditions at the Pine and Tuscarora Remediation Site consist of a layer of glacial lacustrine clay overlying bedrock. As noted in previous sections, the results of the field and laboratory investigations are presented in Appendices A and B, respectively. The physical and engineering properties of the lacustrine clay soils are also summarized in the attached Table 1. A description of the subsurface stratigraphy and pertinent soil properties are presented below.

STRATUM 1

Stratum 1 represents those portions of the lacustrine clay soils that are above the Cayuga Creek channel bottom level of about elevation 561. These soils are

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(1) "Flood Insurance Study for Cayuga Creek," Federal Emergency Management Agency, September 1982.

characterized as stiff to very hard red-brown mottled silty clay. These soils have been hardened by desiccation since they are above the water table, which is at or just above the adjacent Cayuga Creek level. As indicated in Table 1, these soils have water contents depressed toward the plastic limit, relatively high unit weights, high shear strengths (cohesion), low compressibility, and high preconsolidation pressures and overconsolidation ratios. A graph of the water content versus elevation is presented as Figure 3. This graph clearly shows the depressed water level in the Stratum 1 soils above elevation 561.

STRATUM 2

Below about elevation 571, the lacustrine soils become firm to very soft red-brown silty clay. These soils are completely saturated, as they are below the water table, with water contents approaching the liquid limit. The dry unit weights are much lower than for the desiccated zone above the water table. The shear strength of these soils are much lower, ranging from 230 to 1,170 psf, and these soils are relatively compressible. Even at the observed high water contents, the Stratum 2 soils exhibit a significant level of preconsolidation, with overconsolidation ratios ranging from 1.5 to 4.6. The coefficient of permeability of the Stratum 2 soils was determined to be in the range of 2 to 5 x 10^{-8} centimeters per second. Thus, these soils are quite impermeable and would serve as an aquiclude.

STRATUM 3

Typical of most borings drilled at this site, a clayey gravel or gravelly clay zone was encountered just on top of the bedrock. The thickness of this zone ranged from 3 to 5 feet in the three test borings drilled for this project. One representative sample from Stratum 3 was tested in the laboratory and determined to have a low water content of 8.1 percent, and to be nonplastic. This zone would be more permeable than the overlying lacustrine soils and would be much stronger from a strength and compressibility consideration.

STRATUM 4

Stratum 4 is the bedrock beneath the site which is typically encountered at about elevation 551$\pm$. For the three borings drilled as part of this study, bedrock was encountered at elevations ranging between 550.6 and 549.6.

GROUNDWATER

Groundwater measurements were not obtained in the test borings because the natural clays were so impermeable that infiltration of groundwater into the boreholes did not occur during the drilling period. However, the water content profile shown on Figure 3 indicates that groundwater level would be at about or just above the creek level, typically in the 561 to 562 elevation range.

ENGINEERING ANALYSES

The engineering analyses conducted as part of this supplemental investigation are described in this section. Pertinent calculations supporting these engineering analyses are included in Appendices C through E.

EXCAVATION AND EMBANKMENT CONSTRUCTION CONDITIONS

The lacustrine clays of Strata 1 and 2 will have a major impact on the construction conditions. It is expected that the clay soils within Stratum 1 can be readily excavated and placed in compacted fills at approximately their existing natural water content. However, these soils will create a very stiff embankment material that would be subjected to cracking if placed on the soft subgrade soils of Stratum 2 without a reinforcing layer between the two materials. Also, the Stratum 1 soils may be too dry to achieve very low permeabilities such as 1×10^{-7} cm/sec that is required for the on-site cap. Additional compaction tests and permeability tests should be performed to determine the compaction "window" that will provide the desired degree of compaction and low permeability using the on-site soils.

The Stratum 2 very soft clays will have the biggest impact on the proposed site grading work. These soils are completely saturated and relatively week. However, laboratory tests confirmed that these soils are slightly overconsolidated (overconsolidation ratio of 1.5 to 4.6) and with shear strengths of 200 to 500 psf for the softer zones. Pocket penetrometer readings and torvane shear tests in the laboratory indicated that some zones were even weaker. Since these soils are varved silts and silty clays, they would be very susceptible to disturbance by construction equipment and activity. Thus, it is extremely important that hand or very light construction equipment be used over the soft soils until a reinforcing layer has been constructed over the soft soils. For accomplishing this purpose, a coarse aggregate drainage material wrapped in geotextile should be used at the base of any new embankments constructed directly on the Stratum 2 soils. Also, any very loose and saturated creek bottom sediments will require stabilization prior to constructing the overlying embankment. Alternative methods for stabilizing these loose sediments are presented in the Recommendations section.

SLOPE STABILITY

The stability of the slopes proposed for this project were investigated using acomputer slope stability program. The slope stability calculations are presented in Appendix C. For the slope stability analysis, a parametric study was conducted using very conservative shear strengths of 100, 200, and 300 psf for the Stratum 2 soils. It is believed that these soils, on average, will have a shear strength of at least 300 psf. Although the overlying existing clays and/or embankment fills will have shear strengths of at least 1500 psf, strength reductions must be used when placing a stiff clay embankment over a soft, yielding subgrade. A strength ratio of 0.45 between the soft foundation soils and the embankment soils was used in the stability analysis. Final strength parameters used for the stability analysis were 300 psf shear strength for Stratum 1 and 800 psf shear strength for Stratum 2.

The slope stability calculations indicated that the proposed construction would have a factor of safety of at least 1.5. It is important that the embankment slopes be raised carefully and gradually with the use of light compaction equipment on the edge of the embankment slopes. In no cases should heavy trucks or loaded pans be operated near the edge of the embankment slopes.

A parametric study was also conducted to determine how much fill could be stockpiled adjacent to the Cayuga Creek top of slope prior to filling the existing creek. This analysis indicates that if the fill is raised to elevation 576 (2 feet of overfill) and 578 (4 feet of overfill), the corresponding factors of safety would be 1.36 and 1.25. Based upon this calculation, it is Woodward-Clyde Consultants' conclusion that the filling along the top of slope of the existing Cayuga Creek should not exceed elevation 578 and preferably 576.

EMBANKMENT SETTLEMENT

Approximately 4 feet of cap is to be constructed over the existing disposal area. This amount of filling is expected to result in an areal settlement of about 0.5 to 1.5 inches. For the backfilling of the existing channel, fill is to be placed from the existing channel bottom, about elevation 561, up to the top of cap, about elevation 573 to 574. This amount of fill placed over a very soft subgrade will induce significant amounts of settlement. A total of four consolidation tests were performed in the Stratum 2 soils (see Table 1 and Appendix B). These consolidation tests indicate that the preconsolidation pressure of the soft materials is probably in the range of 1200 to 1800 psf as a minimum, and somewhat higher in other zones. An overconsolidation ratio for this material was observed to be in the range of 1.5 to 4.6. For conservatism, it was assumed that the preconsolidation pressure was 1200 psf.

The section analyzed is shown in Appendix D, Settlement Calculations, together with the settlement results. These data indicate that the settlement will be a maximum over the center of the existing creek of about 4.5 inches, tapering to about 1 to 1.5 inches at the edge of the significant fill area. For parametric study purposes, a slightly higher preconsolidation pressure of 1800 was also investigated, and this gave a maximum settlement of about 3 inches in the center of the existing channel. Therefore, based upon the available information, it is estimated that the maximum settlement of the new embankment will be in the range of 3 to 5 inches, tapering to minimal amounts at the edge of the fills. The proposed clay embankments and cap system components can undergo these magnitudes of deformation without impacting their integrity.

Since the compressible layer is relatively thin, averaging about 8 feet, and there is a drainage face on both sides of the compressible layer, the settlement will occur over a fairly short period of time. It is estimated that about 50 percent of the settlement would occur in approximately 2 to 3 months, and that 90 percent would occur in 10 to 13 months.

The impact of these estimated settlements on the design and construction of the slurry wall are negligible. The slurry wall backfill will be appreciably softer and more yielding than either the foundation or embankment soils. Therefore, the slurry wall will readily deform as the embankment foundation soils and embankment materials settle and deform. Thus, no special requirements are required to account for the expected settlement on the slurry wall design or construction.

SEEPAGE CUTOFF

Some concern was raised over whether or not the silty clay soils located in the existing channel bottom and floodplain area would serve as an adequate aquiclude for the slurry wall construction. Three permeability tests were conducted (see Table 1 and Appendix B) to investigate this technical condition. The three test results clearly indicate that the soft natural clays have a permeability well below the permeability desired for a seepage cutoff, 1×10^{-7} cm/sec. The test results showed that the natural clay soils have a permeability in the range of 2 to 5×10^{-8}. Thus, it is concluded that the natural clays will serve as a suitable aquiclude and bottom key material for the proposed slurry wall construction.

SLOPE EROSION PROTECTION

Slope protection for the new channel slopes is a major technical concern. Design calculations to evaluate the impact of design storms on the design of slope protection are included in Appendix E. As shown in these calculations, it was decided to use a 500-year flood event as a design storm for slope protection design. This storm yielded a maximum velocity in the range of 6 to 7 feet per second. Using a shear factor of 2 for a meandering stream, the calculations indicated that the D_{50} size of rip-rap should be about 0.31 foot, or about 4 inches. For comparison purposes, the Corps of Engineers' Procedure For the Hydraulic Design of Flood Control Channels was used, and this indicated that an equivalent stone diameter of about 0.5 to 0.6 foot should be used. These data were then compared with New York State Department of Transportation Specifications for "Stone Filling." It was determined that a light stone fill provides a gradation that best meets the requirements for slope protection. The light stone fill has the following requirements:

Stone Size	Percent of Total By Weight
Lighter than 100 pounds	90 to 100
Larger than 6 inches	50 to 100
Smaller than 1/2 inch	0 to 10

Medium rip-rap, which is the next largest size, would be well above the requirements for slope protection. The medium rip-rap would require that 50 to 100 percent of the stone sizes to be greater than 100 pounds, which is equivalent to a sphere of about 13 inches. These weights and stone sizes are much larger than required. Thus, it was concluded that medium rip-rap would not be appropriate for this project. However, it was concluded that an 18-inch-thick layer of light rip-rap should be placed on the slope of the new channel that is adjacent to the Pine and Tuscarora Remediation Site. The extra thickness (18 inches versus 12 inches for the channel bottom and outside channel slope) would provide added protection for the channel meander as it passes the Pine and Tuscarora Remediation Site.

RECOMMENDATIONS

Recommendations for the design of the Pine and Tuscarora Remediation Site are summarized below.

EMBANKMENT DESIGN

It is recommended that the proposed embankment for filling the existing channel be constructed of the clay materials to be excavated from the new channel. Since the embankment will support no load, and it will be constructed over a soft and yielding subgrade, it is recommended that the embankment be constructed on the wet side of optimum water content determined by the standard Proctor (ASTM D 698) compaction criteria. These compaction criteria will result in an embankment that will be relatively incompressible, but one that will deform substantially without cracking and will also have a very low permeability, less than 10^{-7} cm/sec. Bulk samples of the lacustrine clay should be taken and compaction and permeability tests conducted to determine the suitable water contents for achieving the desired degree of compaction and low permeability.

The bottom of the existing channel contains typically about 12 inches of loose sediments deposited by the normal stream flow. These saturated, loose sediments will require drying and compaction for stabilization or, alternatively, stabilization by mixing with large rip-rap or rock fill, such as "shot rock." It is recommended that two options be considered for stabilizing these materials and containing them in place. One option would be to fill over these materials with about 2 feet of shot rock, allowing the loose sediments to permeate into the voids of the shot rock. The other alternate would be to drain the old channel bottom, after the stream has been relocated into the new channel, and scarify, dry, and recompact the channel bottom sediments. This operation would require very favorable drying weather conditions. The drying and compacting would be compounded by the fact that the natural lacustrine clays beneath the channel bottom sediments are, themselves, quite wet and soft, which would preclude much reworking of these sediments without causing undue disturbance to the underlying lacustrine clay. Therefore, field conditions and weather conditions prevailing at the time that this work is performed will have a major impact on which method would be most feasible.

It is recommended that a coarse aggregate wrapped in geotextile be constructed above the stabilized sediments. A typical detail for the channel bottom stabilization is included as Figure 4. This detail shows the shot rock stabilization alternative.

RIP-RAP DESIGN

It is recommended that light stone fill according to New York State Department of Transportation requirement be used for the rip-rap of the new channel. To provide added protection against the stream meander adjacent to the fill slope of the Pine and Tuscarora Remediation Site, it is recommended that the thickness on the slope next to the site be a minimum of 18 inches, whereas the channel bottom and outside channel slope could be 12 inches thick.

SLURRY WALL DESIGN AND CONSTRUCTION

Based upon the results of this study, there are no basic design or construction changes required for the slurry wall. A recommended change to the berm to be constructed over the slurry wall is recommended, which includes the use of a geogrid and the natural lacustrine clay in lieu of a bentonite-soil mixture previously included in the design drawings. The details of this construction are shown on Figure 5.

For the locations where the slurry wall will cross the existing channel, special measures must be taken to be sure that suitable embankment fill is constructed commencing at the top of the suitable natural soils. This will require removing the 12$\pm$ inches of loose creek sediments and placing a compacted clay embankment directly over the natural, soft lacustrine soils. A construction detail for this special construction is presented on Figure 6. Since the natural clay subgrade will be so soft that it will be impossible to work with mechanical equipment, the initial two lifts of clay embankment should be placed and compacted with hand equipment and light compactive effort. No geotextile reinforcing layers, coarse aggregate or rock fill should be placed within this zone. Because of the critical nature of the construction in this area, the work will have to be scheduled during favorable climatic conditions to achieve the desired results.

LIMITATIONS

All conclusions and recommendations presented in this report are based on the assumption that the subsurface conditions do not deviate appreciably from those disclosed by the test borings, and are subject to confirmation or revision upon our review of the final plans and specifications covering pertinent details of the proposed construction. These conclusions and recommendations are also based on the premise of competent field engineering and inspection during construction.

Tables

TABLE 1

SUMMARY OF SOIL PROPERTIES

Stratum No.	General Description	Boring No.	Sample No.	Sample Depth (ft)	Sample Elev. (ft)	Water[a] Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	Specific Gravity	Dry Unit[a] Weight (pcf)	Cohesion (psf)	Coefficient of Permeability (cm/sec)	Preconsolidation Pressure (psf)	Compression[b] Index	Recompression[b] Index	Overconsolidation Ratio
1	Stiff to Hard Silty Clay	WCC-6	ST-2	5-7	566.8	21.8	45	23	22	--	106.2	2,620					
		WCC-6	ST-3	8-10	563.8	28.2	49	23	26	2.76	94.4	2,950		14,000	0.097	0.018	12.7
		WCC-5	ST-1	2-4	562.5	23.1	40	19	21	2.68	102.2	1,550		3,600	0.113	0.019	12.0
2	Soft to Very Soft Silty Clay	WCC-5	ST-2	5-7	559.5	40.9	43	22	21	2.77	80.7	230		1,800	0.170	0.030	3.2
		WCC-6	ST-5	13-15	558.8	34.6	37	20	17	2.77	87.8	1,170	2.4×10^{-8}	7,400	0.143	0.043	4.6
		WCC-5	ST-3	8-10	556.5	40.0	46	24	22	2.75	82.5		4.3×10^{-8}	1,200	0.122	0.032	1.5
		WCC-6	ST-6	15-17	553.8	36.3	38	22	16	2.75	86.5	580	2.7×10^{-8}	3,300	0.100	0.020	2.2
		P-5	SS-1	2-4	561.6	37.6	42	24	18								
		P-5	SS-2	4-6	559.6	42.2	45	25	20								
		P-5	SS-3	6-8	557.6	42.1	42	24	18								
		P-5	SS-4	8-10	555.6	45.4	41	24	17								
		P-5	SS-5	10-12	553.6	43.1											

[a] Water Content and Dry Unit Weight from Triaxial Test Specimen when available
[b] Unit Strain Basis

Figures

PROJECT No. 2287 - 43

WENDEL ENGINEERS P.C.
consulting engineers/planners/surveyors
LOCKPORT N.Y. - BUFFALO N.Y.

STATE OF NEW YORK — LICENSED LAND SURVEYOR

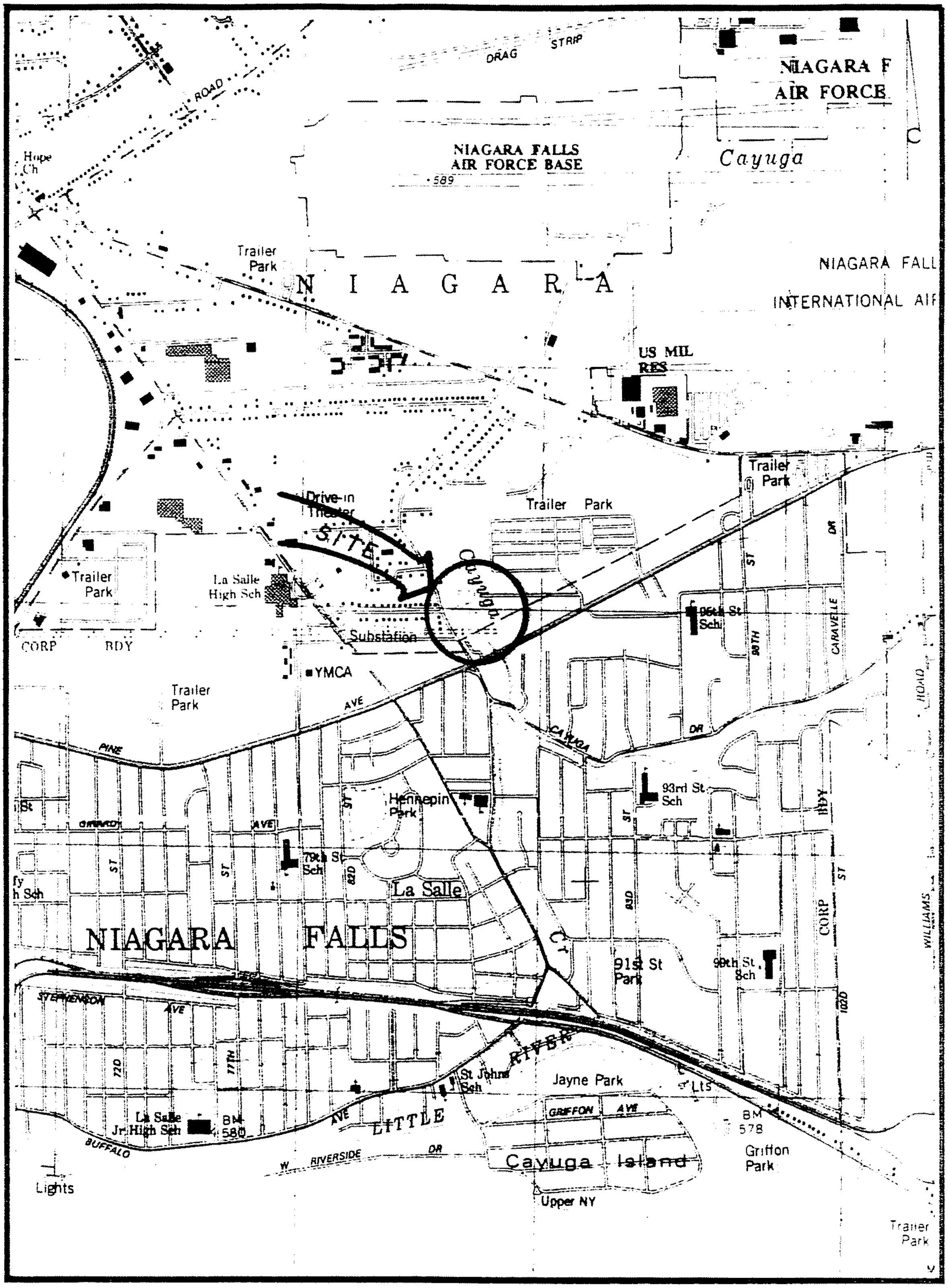
				2	ADDITIONAL LOCATION OF TEST HOLES	M.J.P.	1-29-90	
				1	GENERAL UPDATE & ADDITIONAL LOCATION OF TEST HOLES & MONITORING WELL	M.J.B.	3-2-88	
DWG NO.	REFERENCE DWG	DWG NO.	REFERENCE DWG	REV.	DESCRIPTION	DRAWN	DATE	CHKD.	APPR.

DRAWN DATE P. MAIORANA 11/4/85
CHECKED DATE L. ENNESSER 11/4/85
APPROVED

Olin

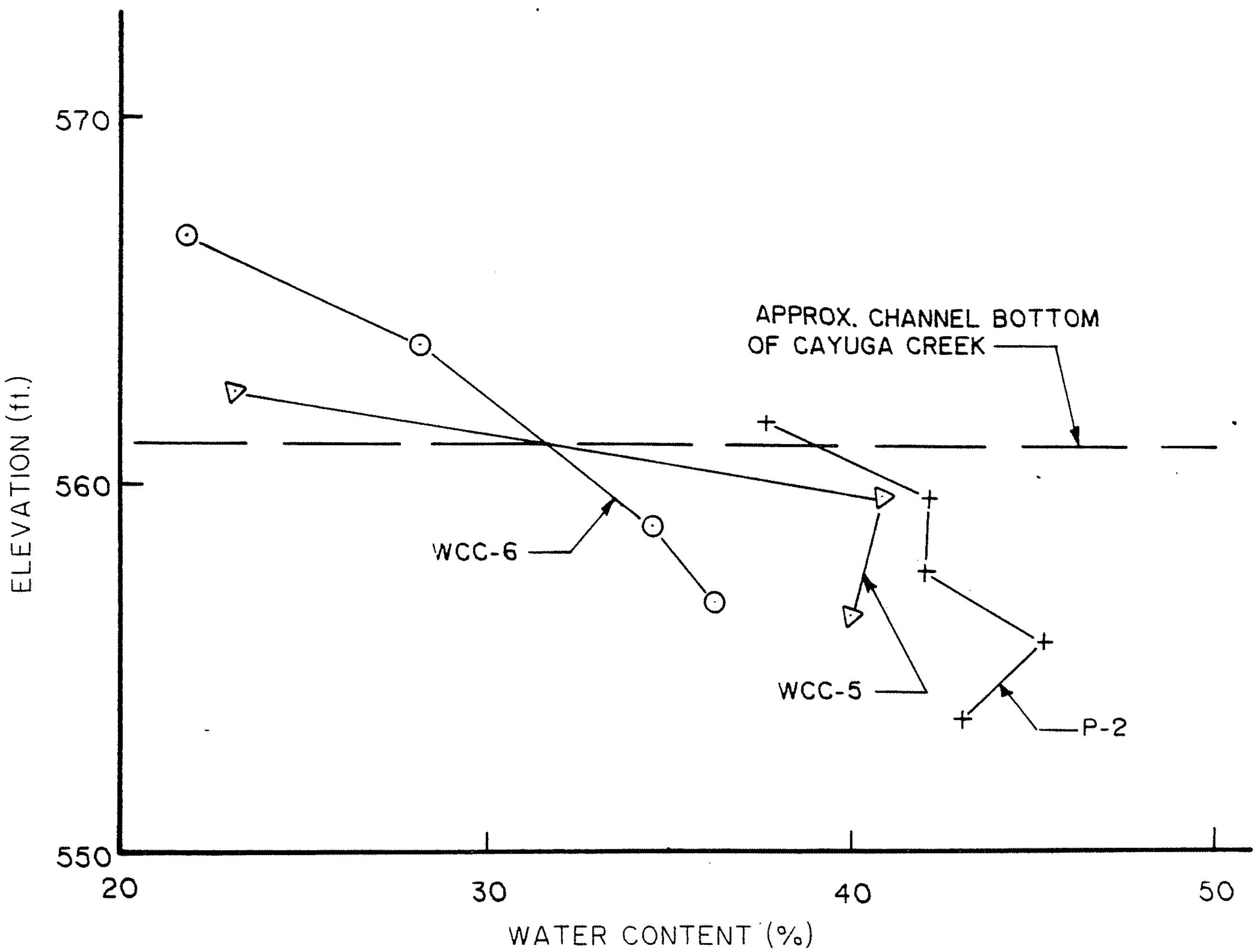
GIBSON SITE, SURVEY
NIAGARA FALLS, NEW YORK

PROJECT NO. | C.A.R. NO. | SCALE 1" = 20' | DRAWING NUMBER | REV.



REGIONAL LOCATION PLAN

FIGURE 1

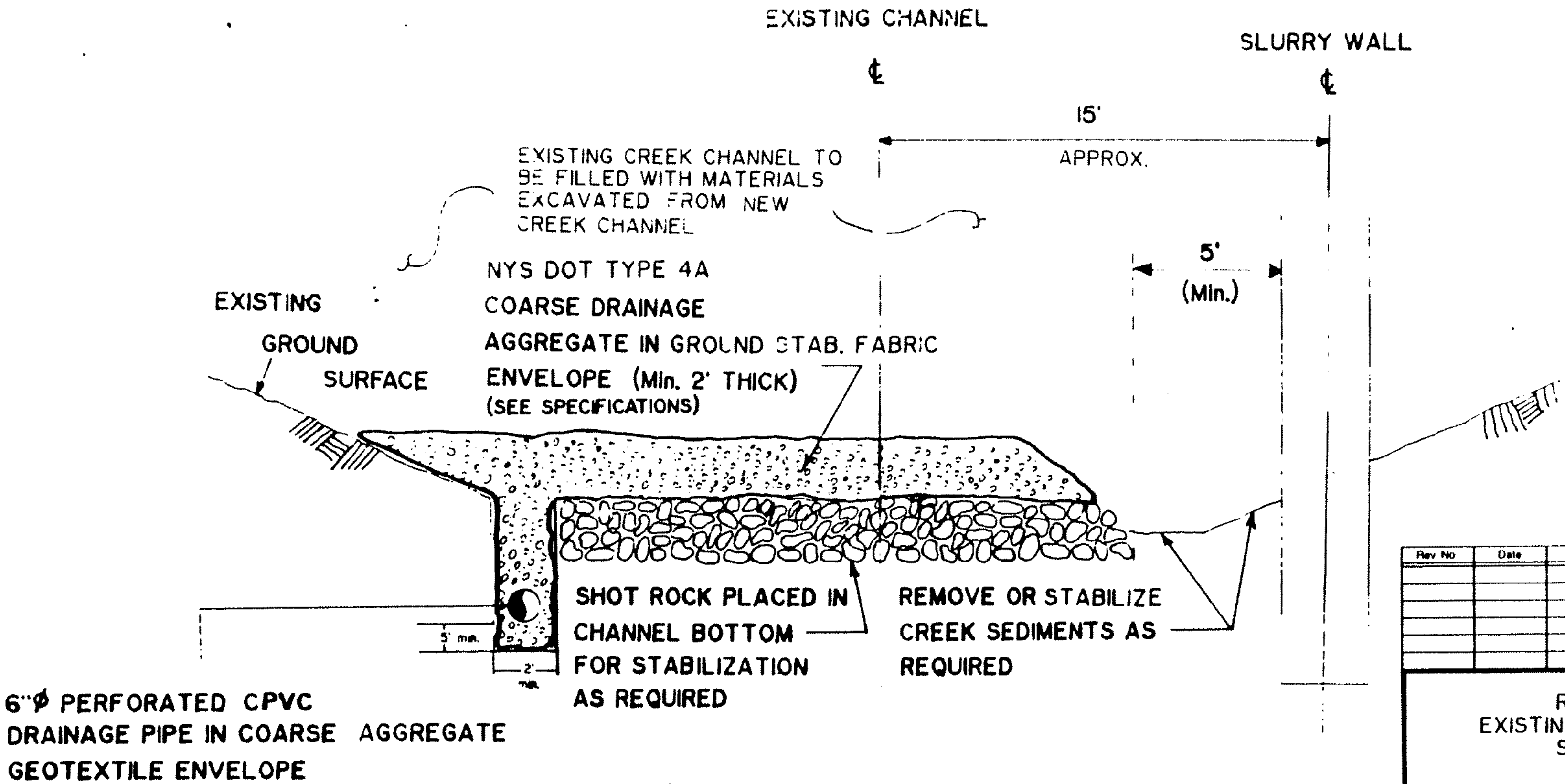


WATER CONTENT
vs.
ELEVATION
OLIN CHEMICALS - GIBSON SITE
NIAGARA FALLS, NEW YORK

Woodward-Clyde Consultants
Consulting Engineers, Geologists and Environmental Scientists

Job No.: 89C2857	Drawing No.	Date: 02/23/90
Drawn by: DNA	Checked by:	
Scale: NONE		FIGURE 3

Rev. No.	Date	Type of Revision	Checked by:



Rev No	Date	Type of Revision	Checked By

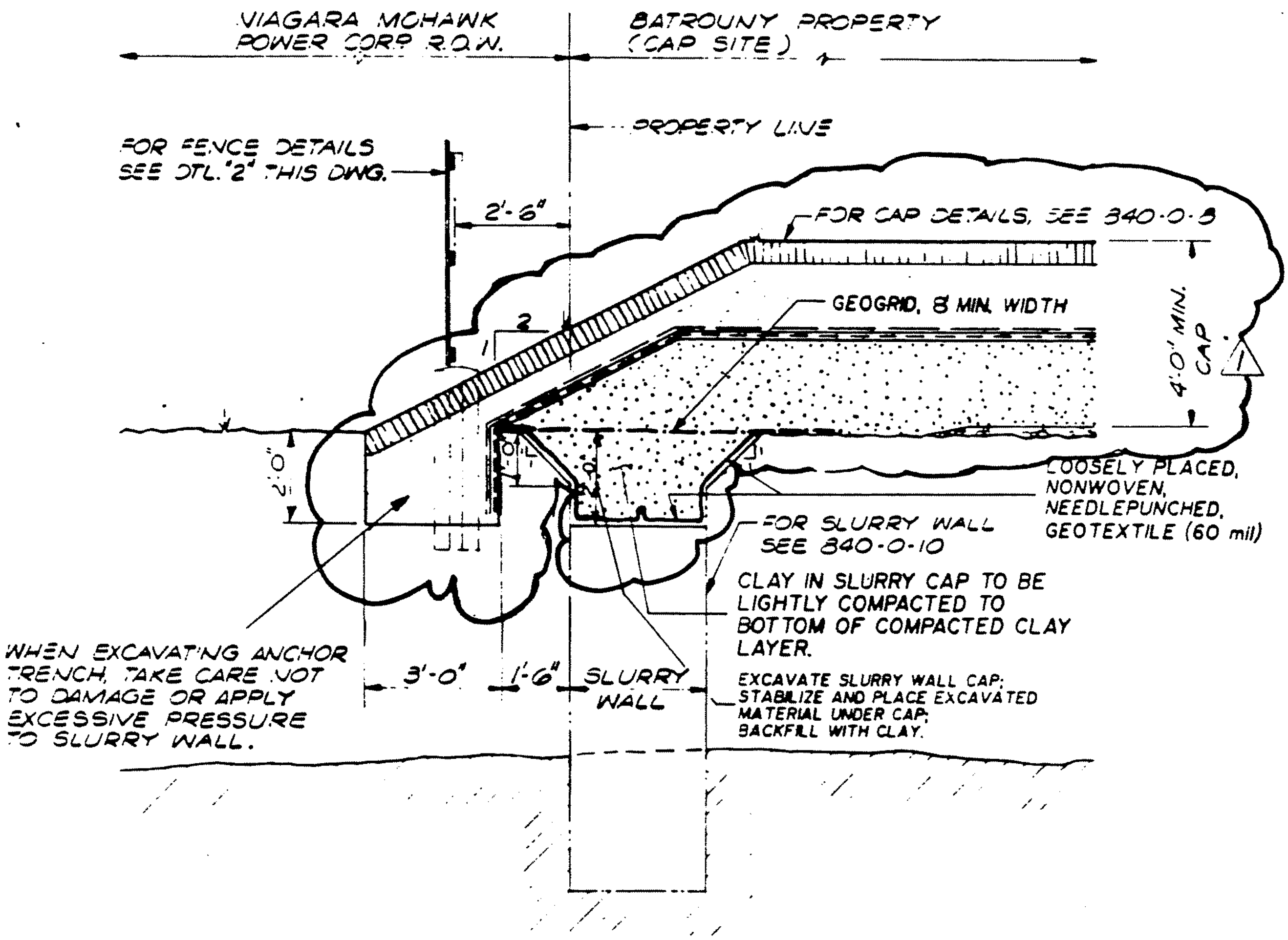
RECOMMENDED
EXISTING CHANNEL BOTTOM
STABILIZATION

OLIN CHEMICALS - GIBSON SITE

Woodward-Clyde Consultants
Consulting Engineers, Geologists and Environmental Scientists

Job No 89C2857	Drawing No	Date 02/23/90	
Drawn by DNA	Checked by	FIGURE 4	
Scale NONE			

Woodward-Clyde Consultants

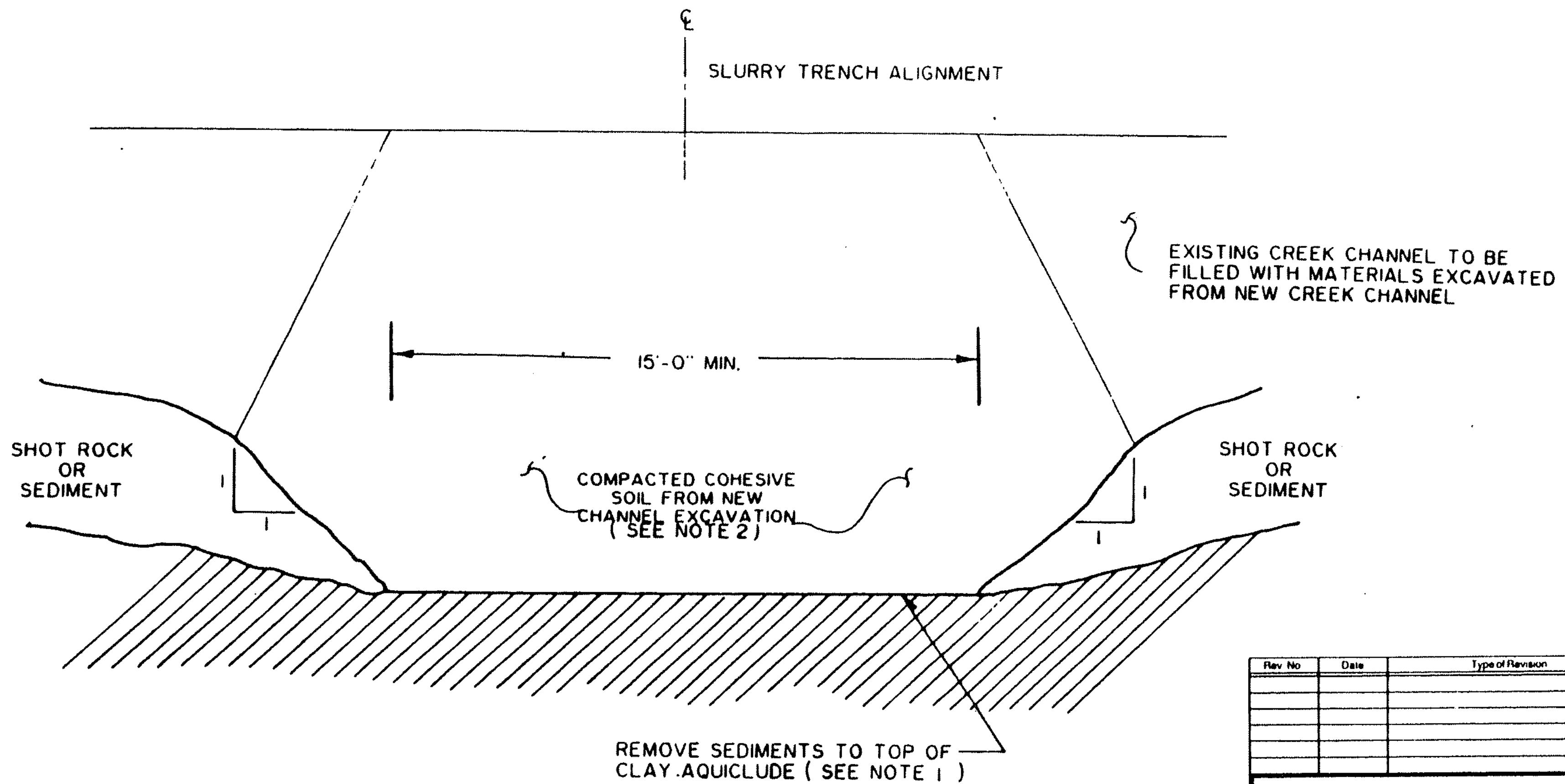


CLAY BERM OVER SLURRY WALL

OLIN CHEMICALS - GIBSON SITE

Drawn by DNA	SCALE IN FEET NONE	Date 02/23/90
Checked by		Job 89C2857

FIGURE 5



Rev No	Date	Type of Revision	Checked By

FOUNDATION PREPARATION
for
SLURRY WALL CROSSING
of
EXISTING CHANNEL
OLIN CHEMICALS - GIBSON SITE

Woodward-Clyde Consultants
Consulting Engineers, Geologists and Environmental Scientists

Job No 89C2857	Drawing No	Date 02/23/90	
Drawn by DNA	Checked by	FIGURE 6	
Scale NONE			

Appendix A

APPENDIX A

FIELD INVESTIGATION

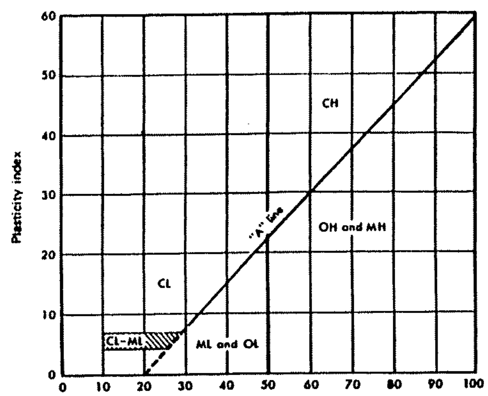
The field investigation consisted of three test borings located as shown on Figure 2. The test borings were drilled by Empire Soils of Hamburg, New York, and the surveying for layout and elevation was conducted by Wendel Engineers of Lockport, New York. The test boring logs are included as pages A-3 through A-6. A Key to Soil Symbols and Terms is included as A-2.

Test borings WCC-5 and WCC-6 were drilled for the purpose of obtaining 3-inch O.D. undisturbed samples of the subsoils for engineering property tests. Nearly continuous undisturbed sampling was performed down to the top of bedrock. Boring P-5 was drilled immediately adjacent to Cayuga Creek so that a comparison of the soils beneath Cayuga Creek could be compared with the results of test boring WCC-5, which is located approximately 65 feet north in the floodplain of Cayuga Creek. The results of both P-5 and WCC-5 were also used to correlate with Boring WCC-6, which was drilled on the higher ground east of the Pine and Tuscarora Remediation Site. All soil samples were delivered to Woodward-Clyde Consultants' Plymouth Meeting, Pennsylvania laboratory for further visual classification and laboratory testing.

KEY TO SOIL SYMBOLS AND TERMS

Terms used in this report for describing soils according to their texture or grain size distribution are in accordance with the Unified Soil Classification System, as described in Technical Memorandum No. 3-357, Waterways Experiment Station, March 1953.

Major Divisions			Group symbols		Typical names	Laboratory classification criteria		
Coarse-grained soils (More than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixture, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 per cent ...... GW, GP, SW, SP; More than 12 per cent ...... GM, GC, SM, SC; 5 to 12 per cent ...... *Borderline* cases requiring dual symbols**	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM*	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM*	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols.
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. > than 7	
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity			
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
Highly organic soils			Pt		Peat and other highly organic soils			



Liquid limit

Plasticity Chart

*Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

**Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example: GW-GC, well-graded gravel-sand mixture with clay binder.

Material	Particle Size: mm	Particle Size: Sieve Size
Silt or Clay	<0.074	<#200
Sand: Fine	0.074 To 0.42	#200 To #40
Sand: Med.	0.42 To 2.00	# 40 To #10
Sand: Coarse	2.00 To 4.76	# 10 To # 4

Material	Particle Size: mm	Particle Size: Sieve
Gravel: Fine	4.76 To 19.1	#4 To 3/4 In.
Gravel: Coarse	19.1 To 76.2	3/4 In To 3 In.
Cobbles	76.2 To 304.8	3 In. To 12 In.
Boulders	304.8 To 914.4	12 In. To 36 In.

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS (major portion retained on No. 200 sieve): Includes (1) clean gravels and (2) silty or clayey gravels and sands. Condition is rated according to relative density[1] as determined by laboratory tests or standard penetration resistance tests.

Descriptive Term	Relative Density
Very loose	0 to 15%
Loose	15 to 35%
Medium dense	35 to 65%
Dense	65 to 85%
Very dense	85 to 100%

FINE GRAINED SOILS (major portion passing No. 200 sieve): Includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as indicated by penetrometer readings or by unconfined compression tests.

Descriptive Term	Unconfined Compression Strength, tons/sq. ft.
Very soft	less than 0.25
Soft	0.25 to 0.50
Firm	0.50 to 1.00
Stiff	1.00 to 2.00
Very stiff	2.00 to 4.00
Hard	4.00 and higher

TEST AND SAMPLE IDENTIFICATION

15 — 15 – The number of blows (15) of a 140-pound hammer falling 30 inches used to drive a 2" O. D. split-barrel sampler for the last 12 inches of penetration.

50/2 — 50/2 – Number of blows (50) used to drive the split-barrel a certain number of inches (2).

P – Thin-wall tube sample.

P/250 — P250 – Thin-wall tube pushed hydraulically, using a certain pressure (250 psi) to push the last 6 inches.

C_t – Denison or Pitcher-Type – core-barrel sample.

Ps — Ps – Piston sample.

A – Auger sample.

BX – Rock cored with BX core barrel, which obtains a 1-5/8" diameter core.

NX/65 — NX – Rock cored with NX core barrel, which obtains a 2-1/8" diameter core.

65% – Percentage (65) of rock core recovered.

20 — 20% – Rock Quality Designation (RQD)[2]

VS — VS – Vane Shear Test.

Sample Recovered / Sample Not Recovered

- C – Consolidation and specific gravity tests.
- D – Maximum & minimum density.
- DS – Direct Shear test.
- G – Specific gravity test.
- K – Permeability test.
- M – Mechanical (sieve or hydrometer) analysis.
- T – Triaxial compression test
- U – Unconfined compression test.
- W – Unit weight & natural moisture content.
- X – Special tests performed – see Laboratory test results.

$\}$ Laboratory Test Performed

(2) $RQD = \frac{\sum \text{Core Segments} \geq 4 \text{ Inches}}{\text{Core Interval}} \times 100$

Where Segmentation Is Not Caused By Drilling Effects

(1) ASTM 2049-69

LOG of BORING No. P-5

Sheet 1 of 1

DATE 1/24/90 SURFACE ELEVATION 564.6 LOCATION See Figure 2

DEPTH, ft.	SAMPLING RESISTANCE	SAMPLE TYPE	DESCRIPTION	STRATUM ELEVATION	POCKET PENETROMETER	FIELD TEST	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	OTHER TESTS
0			Very soft brown organic clay and silt							
	WOH	SS		562.0	0.0		37.6	42	24	
			Very soft to soft red/brown clay							
5	1	SS			0.2		42.2	45	25	
	4	SS			0.0		42.1	42	24	
	WOH	SS			0.0		45.4	41	24	
10	2	SS		553.6			43.1			
	19	SS	Very stiff red/brown coarse to fine gravelly clay, trace cobbles		3.5		8.1	NP	NP	
				550.1						
15			NOTES: (1) Pocket Penetrometer Resistance in tons per square foot, an indication of unconfined compressive strength (2) Auger refusal at 14.5 feet (3) WOH = Weight of hammer (4) SS = Standard Split-Spoon Sampler							
20										
25										
30										
35										
40										

Completion Depth: 14.5 Ft.

Project No.: 89C2857

Project Name: Olin Pine & Tuscarora Remediation Site

Drilling Method: Hollow Stem Augers

Water Depth: ______ ft., After ______ hrs.

______ ft., After ______ hrs.

______ ft., After ______ hrs.

______ ft., After ______ hrs.

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LOG of BORING No. WCC-5

Sheet 1 of 1

DATE 1/24/90 SURFACE ELEVATION 565.5 LOCATION See Figure 2

DEPTH, ft.	SAMPLING RESISTANCE	SAMPLE TYPE	DESCRIPTION	STRATUM ELEVATION	POCKET PENETROMETER	FIELD TEST	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	OTHER TESTS
0			Very soft black organic clay (6")	565.0						
	P/0	ST	Stiff to very stiff gray/tan clay, trace silt		1.3- 2.3		23.1 25.1	40	19	Tx C
5				560.5						
	P/0	ST	Very soft red/brown clay		0.0		40.9 40.1	43	22	Tx C
	P/0	ST			0.0		40.0 39.8	46	24	K C
10				555.5						
			Stiff to very stiff red/brown coarse to fine gravelly clay, trace cobbles							
15	P	ST		550.6						
20										
25										
30										
35										
40										

NOTES:
(1) Pocket Penetrometer Resistance in tons per square foot, an indication of unconfined compressive strength
(2) Auger refusal at 14.9 feet
(3) ST = Standard 3-inch O.D. Shelby Tube Sampler
(4) P = Shelby Tube resistance in pounds per square inch

Completion Depth: 14.9 Ft. Water Depth: ____ ft., After ____ hrs.
Project No.: 89C2857 ____ ft., After ____ hrs.
Project Name: Olin Pine & Tuscarora Remediation Site ____ ft., After ____ hrs.
Drilling Method: Hollow Stem Augers ____ ft., After ____ hrs.

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LOG of BORING No. WCC-6

DATE 1/23/90 SURFACE ELEVATION 572.8 LOCATION See Figure 2

DEPTH, ft.	SAMPLING RESISTANCE	SAMPLE TYPE	DESCRIPTION	STRATUM ELEVATION	POCKET PENETROMETER	FIELD TEST	WATER CONTENT, %	LIQUID LIMIT, %	PLASTIC LIMIT, %	OTHER TESTS
0			Soft to firm light brown, with dark brown and gray mottles, silty clay							
	P/ 200-250	ST			0.8					
5				567.8						
	P/ 175-200	ST	Firm to hard red/brown, with tan and gray mottles, clay		>4.5		21.8	45	23	Tx
	P/90	ST			2.3		28.2	49	23	Tx
							26.6			C
10										
	P/0	ST			1.2					
	P/0	ST			0.0		34.6	37	20	Tx
				557.8			30.7			C,K
15	P/0	ST	Very soft red/brown clay, trace fine gravel		0.0		36.3	38	22	Tx
							38.0			C,K
	P/0	ST			0.0					
20				552.8						
			Dense to very dense red/brown clayey fine gravel, trace coarse gravel	549.6						
25										
30										
35										
40										

NOTES:
(1) Pocket Penetrometer Resistance in tons per square foot, an indication of unconfined compressive strength
(2) Auger refusal at 23.2 feet
(3) ST = Standard 3-inch O.D. Shelby Tube Sampler
(4) P = Shelby Tube resistance in pounds per square inch

Completion Depth: 23.2 Ft.
Project No.: 89C2857
Project Name: Olin Pine & Tuscarora Remediation Site
Drilling Method: Hollow Stem Augers

Water Depth: ____ ft., After ____ hrs.
____ ft., After ____ hrs.
____ ft., After ____ hrs.
____ ft., After ____ hrs.

Appendix B

APPENDIX B
LABORATORY INVESTIGATION

Physical and engineering property tests were conducted on both disturbed and undisturbed samples obtained from the Pine and Tuscarora Remediation Site. Physical property tests included natural water content, liquid and plastic limits, specific gravity, unit weight determinations, and grain size distribution by hydrometer analysis. The results of tests conducted on the undisturbed samples are summarized on page B-2. A summary of the results for all tests is included as page B-3. The results of the hydrometer analysis on three samples is included as page B-4.

The engineering property tests consisted of triaxial unconsolidated, undrained shear tests to determine the shear strength, consolidation tests to evaluate compressibility, and permeability tests to evaluate the coefficient of permeability. The triaxial shear tests are presented on pages B-5 through B-10. The shear strength, also known as cohesion, is equal to one-half of the maximum deviator stress. In the laboratory, torvane shear tests were also conducted on the samples as a second method to measure the shear strength of the soils. The results of these tests are included with the summary presented on page B-2. The consolidation test results are presented on pages B-11 through B-16. The results are presented giving both the unit strain versus log pressure curves as well as strain versus log time curves. The results of three permeability tests are summarized on the Summary of Laboratory Test Results, page B-3.

PROJECT: Olin-Gibson Site

PROJECT No. 89C2857

SUMMARY OF LABORATORY TEST RESULTS

BORING and SAMPLE No	DEPTH (feet)	CLASSIFICATION	SPECIAL TESTS (a)	NATURAL WATER CONTENT (%)	ATTERBERG LIMITS		UNCON COMPRESS		UNIT DRY WGT (pcf)	SPECIFIC GRA	GRAIN SIZE		OPT. MOIST	CONSOLID	TRIAXIAL			
					LIQUID LIMIT	PLASTIC LIMIT	STRESS (tsf)	STRAIN (%)			SIEVE	HYDR.			U.U.	CIU	CELL PRESSURE (psi)	BACK PRESSURE (psi)
P-5 SS-1	2-4	CL with organics in top 2 inches		37.6	42	24												
SS-2	4-6	CL		42.2	45	25												
SS-3	6-8	CL		42.1	42	24												
SS-4	8-10	CL		45.4	41	24												
SS-5	10-12	CL		43.1														
SS-6	12-14	GC		8.1	NP	NP												
WCC-5 ST-1	2-4	CL		25.1	40	19			102.2 @23.1%	2.68		*		*	*			
ST-2	5-7	CL		40.1	43	22			80.7 @40.9%	2.77		*		*	*			
ST-3	8-10	CL	4.3 x 10^{-8}	39.8	46	24			82.5 @40.0%	2.75				*				
WCC-6 ST-1	2-4																	
ST-2	5-7	CL		21.8	45	23			106.2 @21.8%					*				
ST-3	8-10	CL		26.6	49	23			94.4 @28.2%	2.76		*		*	*			
ST-5	13-15	CL	2.4 x 10^{-8}	30.7	37	20			87.8 @34.6%	2.77				*	*			
ST-6	15-17	CL	2.7 x 10^{-8}	30.9	38	22			86.5 @36.3%	2.75				*	*			

(a) Coefficient of Permeability (cm/sec)

* See Test Curves

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89C2857
Olin-Gibson Site

SUMMARY OF TEST RESULTS FOR UNDISTURBED SAMPLES

Boring No.	Sample Depth	Type of Test	M.C. %	γ_d pcf	LL/PL	$(\sigma_1-\sigma_3)f$ tsf	Su tsf	k cm/sec	$\bar{\sigma}_c$ tsf	σ_c tsf
WCC-5	2-4	-	-	-	-	-	-	-	-	-
ST1	2.30	Trim.	25.1	-	-	-	-	-	-	-
	2.35	Consol.	26.5	97.5	40/19	-	-	-	-	-
	2.55	UU	23.1	102.2	-	1.55	-	-	-	0.22
WCC-5	5-7	-	-	-	-	-	-	-	-	-
ST-2	5.45	Trim.	40.1	-	-	-	-	-	-	-
	5.60	Consol.	49.6	73.6	43/22	-	-	-	-	-
	6.10	UU	40.9	80.7	-	0.23	-	-	-	0.29
	6.50	TV	41.0	-	-	-	0.18	-	-	-
WCC-5	8-10	-	-	-	-	-	-	-	-	-
ST-3	8.35	Trim.	39.8	-	-	-	-	-	-	-
	8.40	Consol.	40.0	82.5	-	-	-	-	-	-
	8.65	TV	46.2	-	-	-	0.13	-	-	-
	8.65	Perm.	47.0	75.4	46/24	-	-	4.3×10^{-8}	0.36	-
	8.75	TV	47.0	-	-	-	0.10	-	-	-
WCC-6	5-7	-	-	-	-	-	-	-	-	-
ST-2	5.70	UU	21.8	106.2	45/23	2.62	(Slickensided)			0.29
	6.10	TV	21.6	-	-	-	2.25	-	-	-
	6.20	PP	21.6	-	-	4.5	-	-	-	-
WCC-6	8-10	-	-	-	-	-	-	-	-	-
ST-3	8.30	TV	26.6	-	-	-	1.5	-	-	-
	8.30	PP	26.6	-	-	4.0	-	-	-	-
	8.60	Consol.	26.1	100.9	49/23	-	-	-	-	-
	9.20	UU	28.2	94.4	-	2.95	(Slickensided)			0.36
WCC-6	13-15	-	-	-	-	-	-	-	-	-
ST-5	13.30	TV	30.7	-	-	-	0.65	-	-	-
	13.60	Consol.	34.1	89.6	37/20	-	-	-	-	-
	14.00	UU	34.6	87.8	-	1.17	(Slickensided)			0.58
	14.40	Perm.	33.1	89.5	-	-	-	2.38×10^{-8}	0.58	-
WCC-6	15-17	-	-	-	-	-	-	-	-	-
ST-6	15.35	Trim.	30.9	-	-	-	-	-	-	-
	15.45	Consol.	30.7	94.8	38/22	-	-	-	-	-
	15.95	UU	36.3	86.5	-	0.58	-	-	-	0.72
	16.60	Perm.	38.0	84.0	-	-	0.38	2.7×10^{-8}	0.72	-

where:

- M.C. = natural water content
- γ_d = initial dry density
- LL/PL = liquid and plastic limits
- $(\sigma_1-\sigma_3)$ = maximum deviatory stress
- Su = shear stress from Torvane (undrained)
- k = coefficient of permeability
- $\bar{\sigma}_c$ = effective stress used for k
- σ_c = confining pressure used for UU test
- TV = torvane reading
- PP = pocket penetrometer

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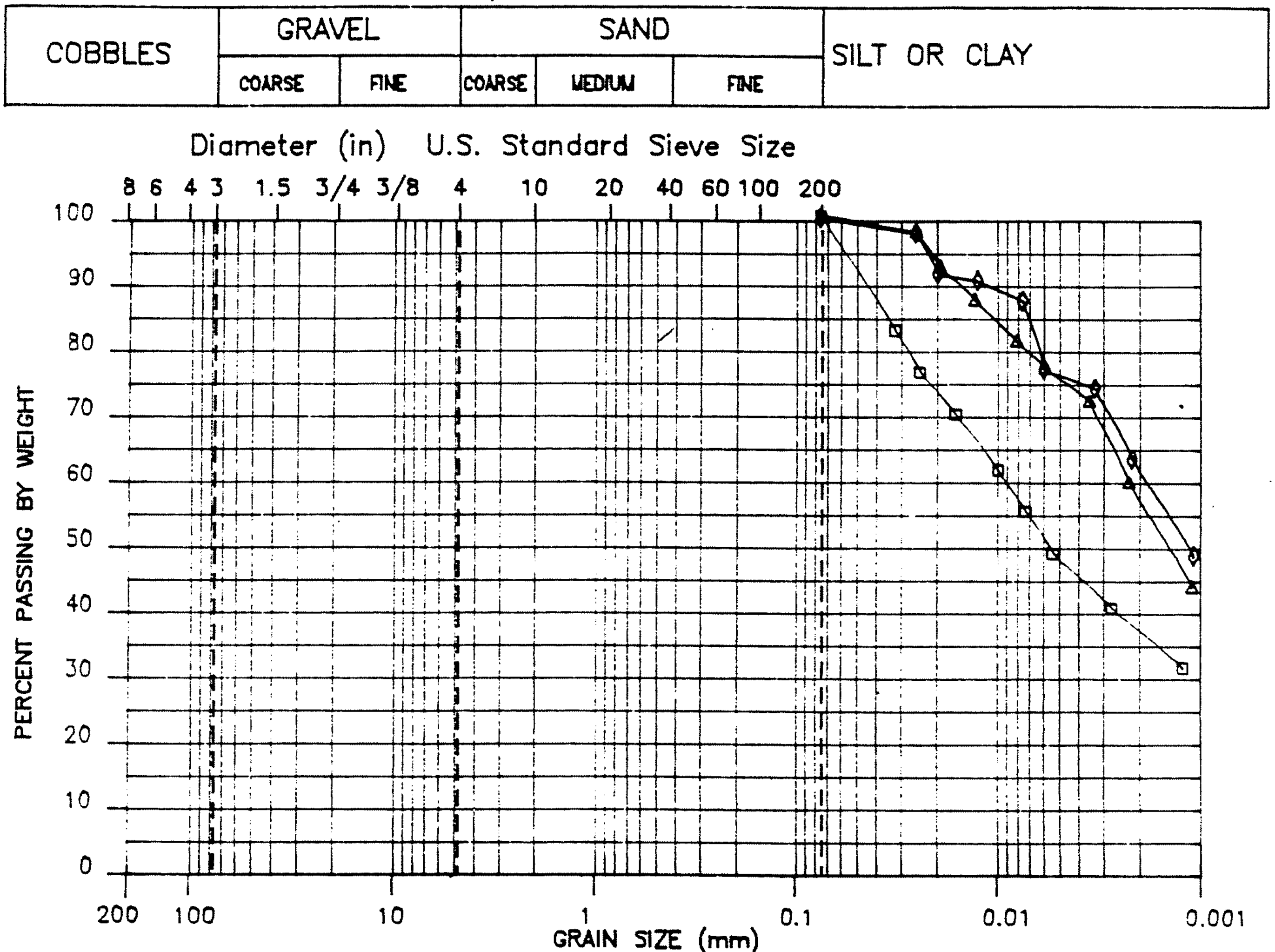
PLYMOUTH MEETING LABORATORY
PARTICLE-SIZE DISTRIBUTION



JOB NUMBER : 89C2857
JOB NAME : OLIN-GIBSON SITE

SYM	BORING#	SAMPLE#	DEPTH	DESCRIPTION	W(%)	WL (%)	WP (%)
□	WCC-5	ST-1	2.55'	BRN-GRAY SILTY CLAY. (CL)	25.1	40	19
Δ	WCC-5	ST-2	6.10'	BRN-GRAY SILTY CLAY. (CL)	40.1	43	22
◊	WCC-6	ST-3	9.15'	GRAY-BRN SILTY CLAY. (CL)	26.6	49	23

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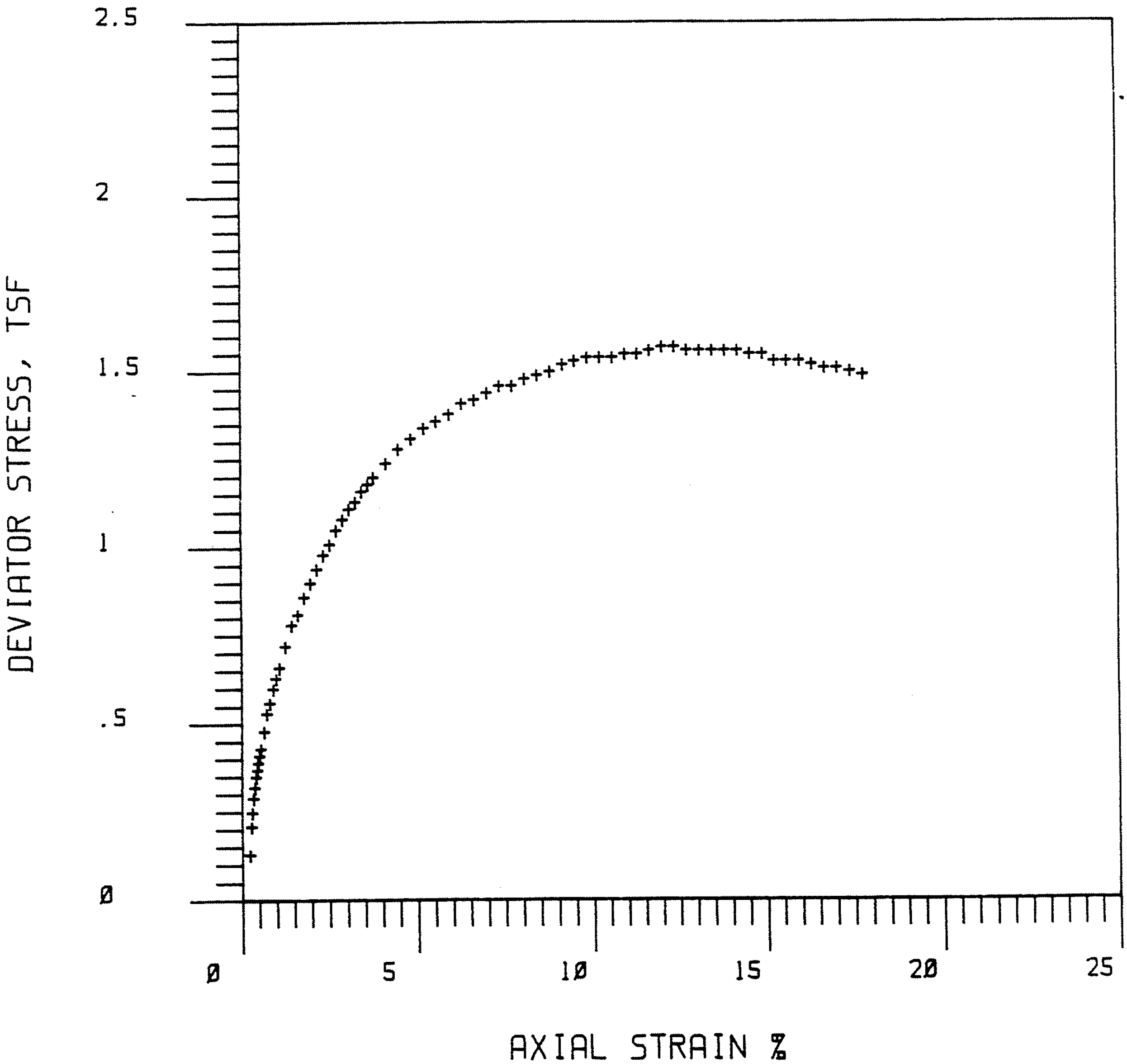
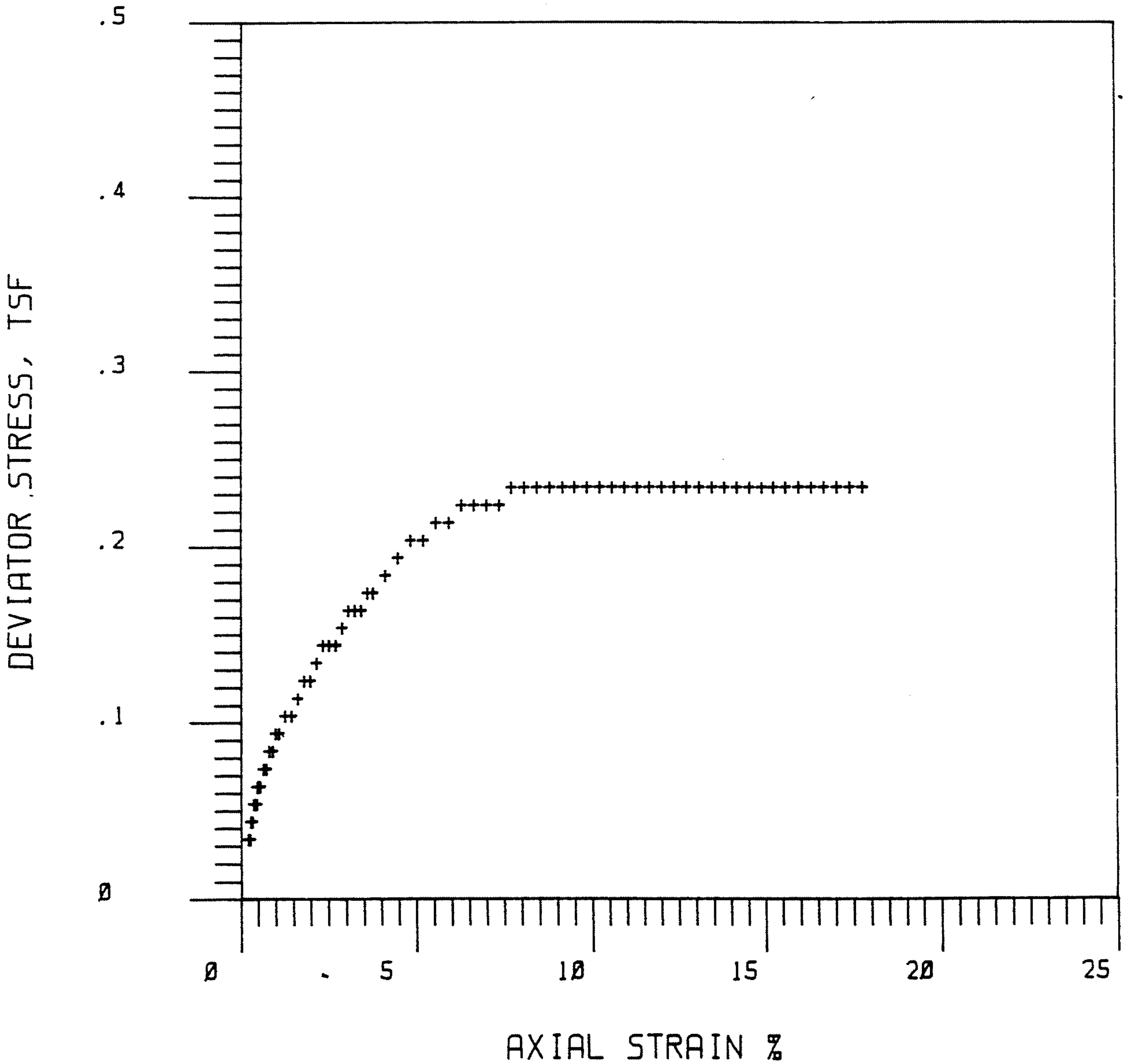
UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
JOB NO. 89C2857
BORING NO. WCC-5
SAMPLE NO. ST-1
SAMPLE DEPTH 2.55 FT.
NATURAL WATER CONTENT 23.09 %
DRY DENSITY 102.22 PCF
MAX DEVIATOR STRESS 1.55 TSF
STRAIN AT FAILURE 12.2 %
EFFECTIVE CONFINING PRESSURE: 0.216 TSF



WOODWARD-CLYDE CONSULTANTS
PLYMOUTH MEETING LAB

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
JOB NO. 89C2857
BORING NO. WCC-5
SAMPLE NO. ST-2
SAMPLE DEPTH 6.10 FT.
NATURAL WATER CONTENT 40.93 %
DRY DENSITY 80.72 PCF
MAX DEVIATOR STRESS .23 TSF
STRAIN AT FAILURE 17.94 %
EFFECTIVE CONFINING PRESSURE: 0.288 TSF



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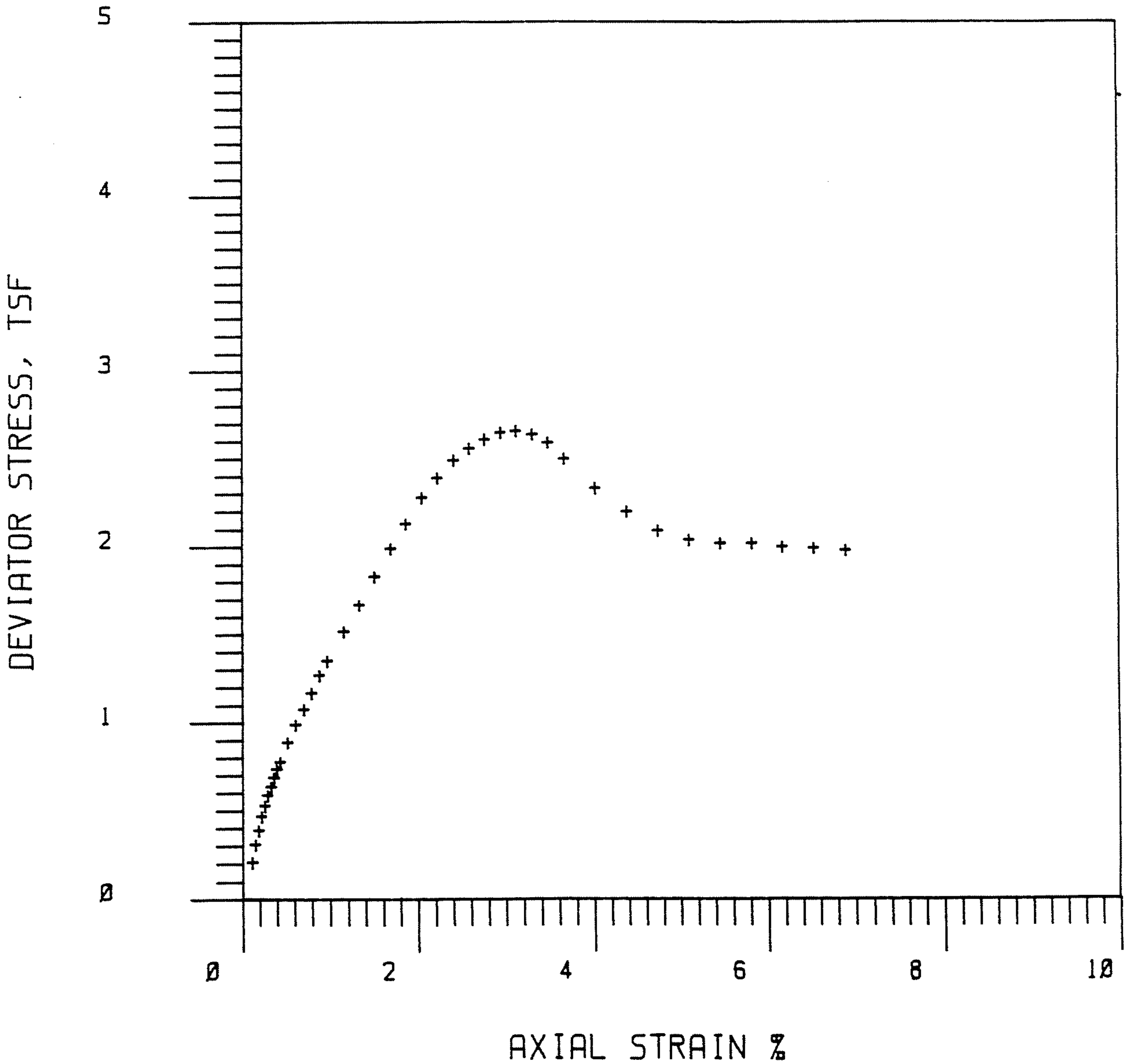
PLYMOUTH MEETING LAB

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
JOB NO. 89C2857
BORING NO. WCC-6
SAMPLE NO. ST-2
SAMPLE DEPTH 5.70 FT.
NATURAL WATER CONTENT 21.84 %
DRY DENSITY 106.18 PCF
MAX DEVIATOR STRESS 2.62 TSF
STRAIN AT FAILURE 3.05 %
EFFECTIVE CONFINING PRESSURE: 0.288 TSF



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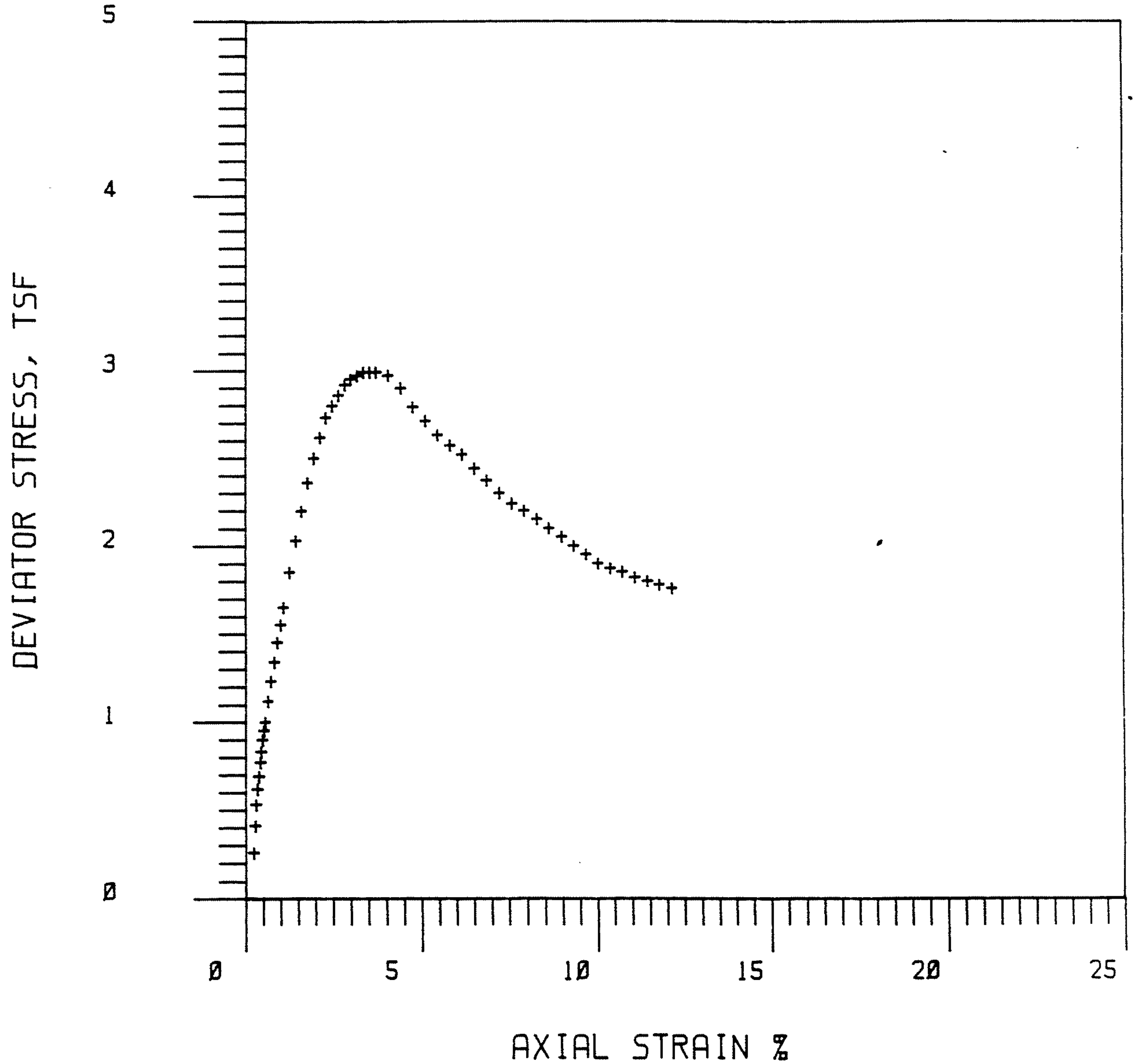
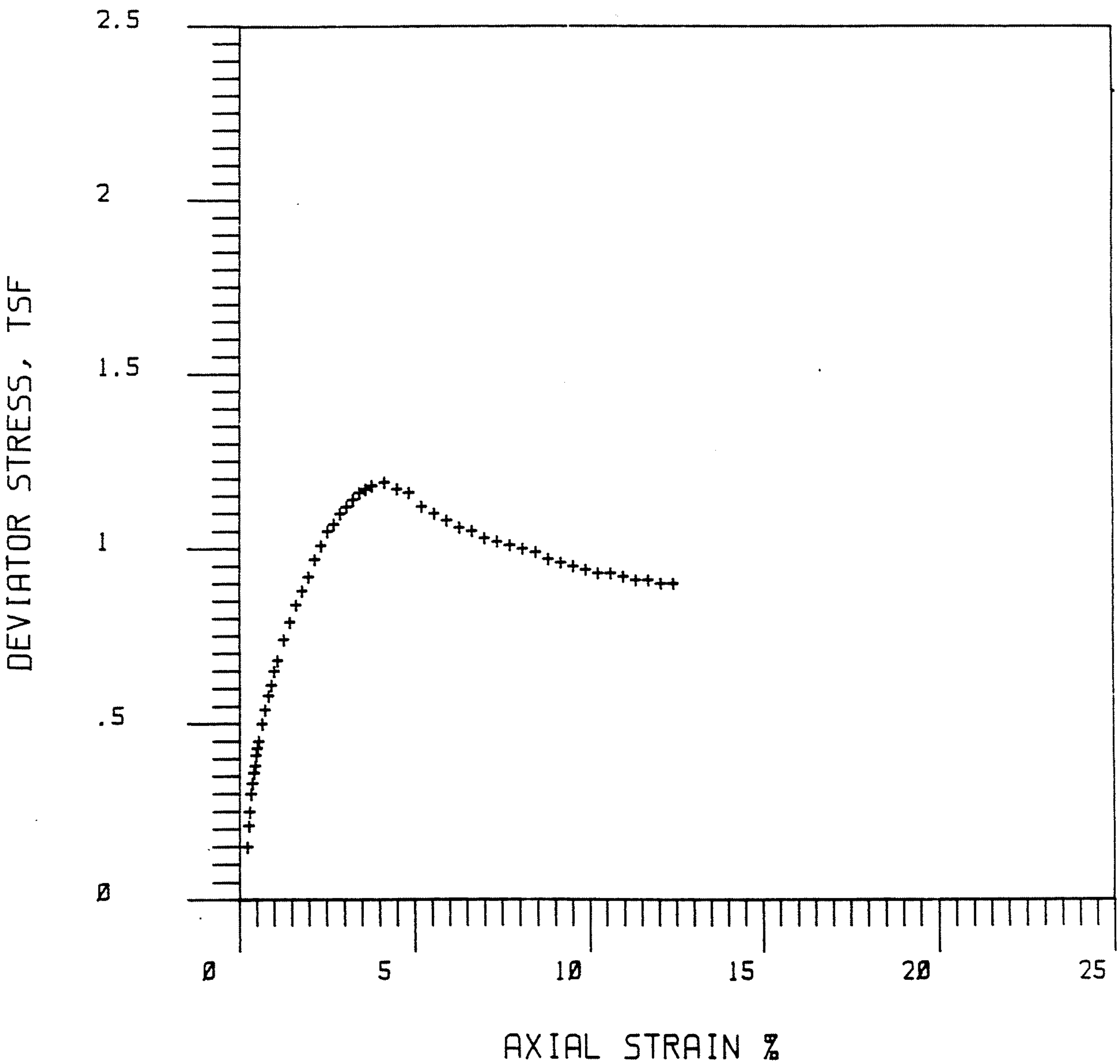
PLYMOUTH MEETING LAB

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
JOB NO. 89C2857
BORING NO. WCC-6
SAMPLE NO. ST-3
SAMPLE DEPTH 9.15 FT.
NATURAL WATER CONTENT 28.18 %
DRY DENSITY 94.37 PCF
MAX DEVIATOR STRESS 2.95 TSF
STRAIN AT FAILURE 3.51 %
EFFECTIVE CONFINING PRESSURE: 0.36 TSF



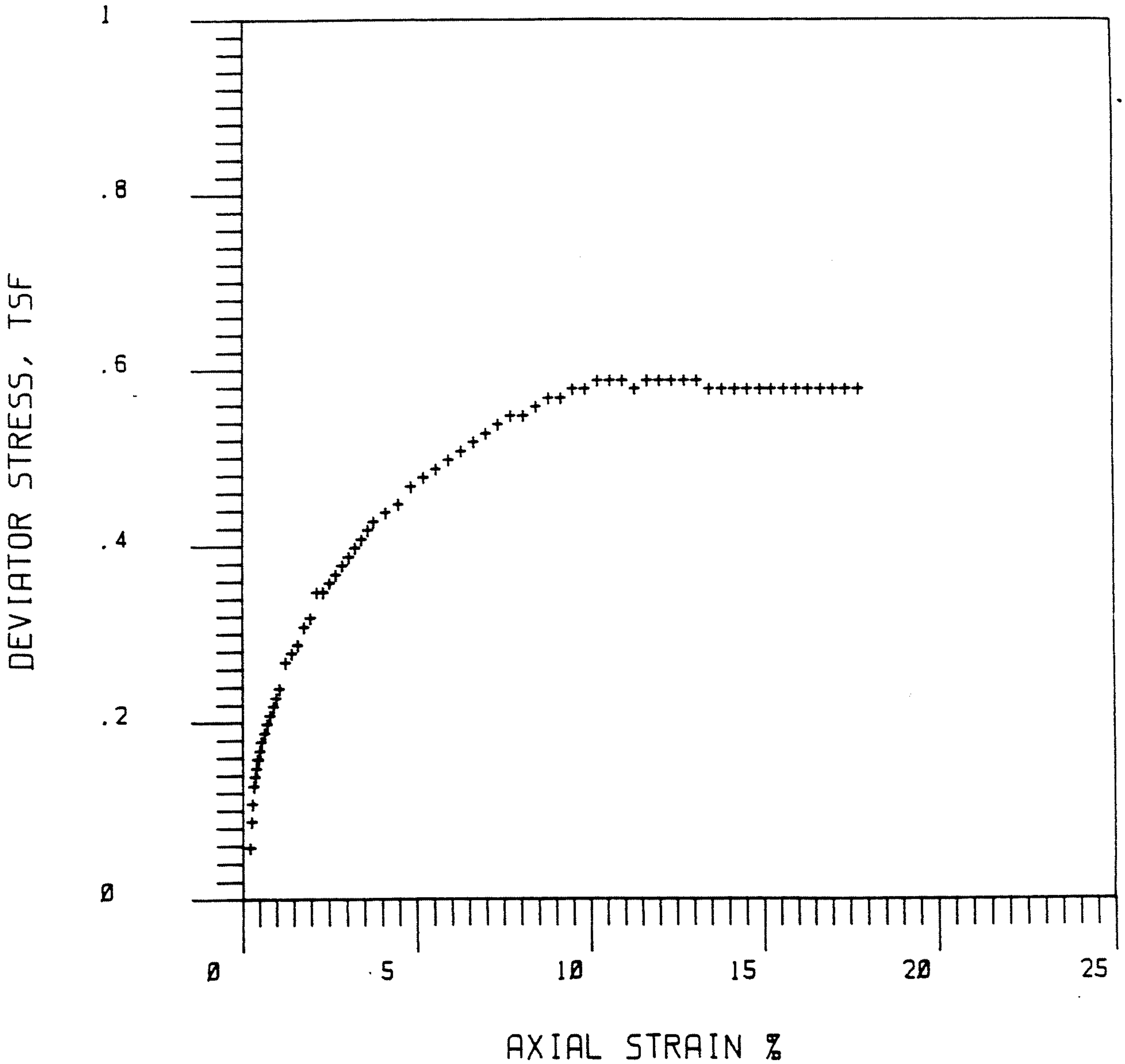
WOODWARD-CLYDE CONSULTANTS
PLYMOUTH MEETING LAB

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
JOB NO. 89C2857
BORING NO. WCC-6
SAMPLE NO. ST-5
SAMPLE DEPTH 14.Ø FT.
NATURAL WATER CONTENT 34.59 %
DRY DENSITY 87.83 PCF
MAX DEVIATOR STRESS 1.17 TSF
STRAIN AT FAILURE 3.95 %
EFFECTIVE CONFINING PRESSURE: Ø.576 TSF



WOODWARD-CLYDE CONSULTANTS
PLYMOUTH MEETING LAB

UNCONSOLIDATED-UNDRAINED TRIAXIAL TEST
JOB NO. 89C2857
BORING NO. WCC-6
SAMPLE NO. ST-6
SAMPLE DEPTH 15.95 FT.
NATURAL WATER CONTENT 36.33 %
DRY DENSITY 86.5 PCF
MAX DEVIATOR STRESS .58 TSF
STRAIN AT FAILURE 12.91 %
EFFECTIVE CONFINING PRESSURE: Ø.72 TSF

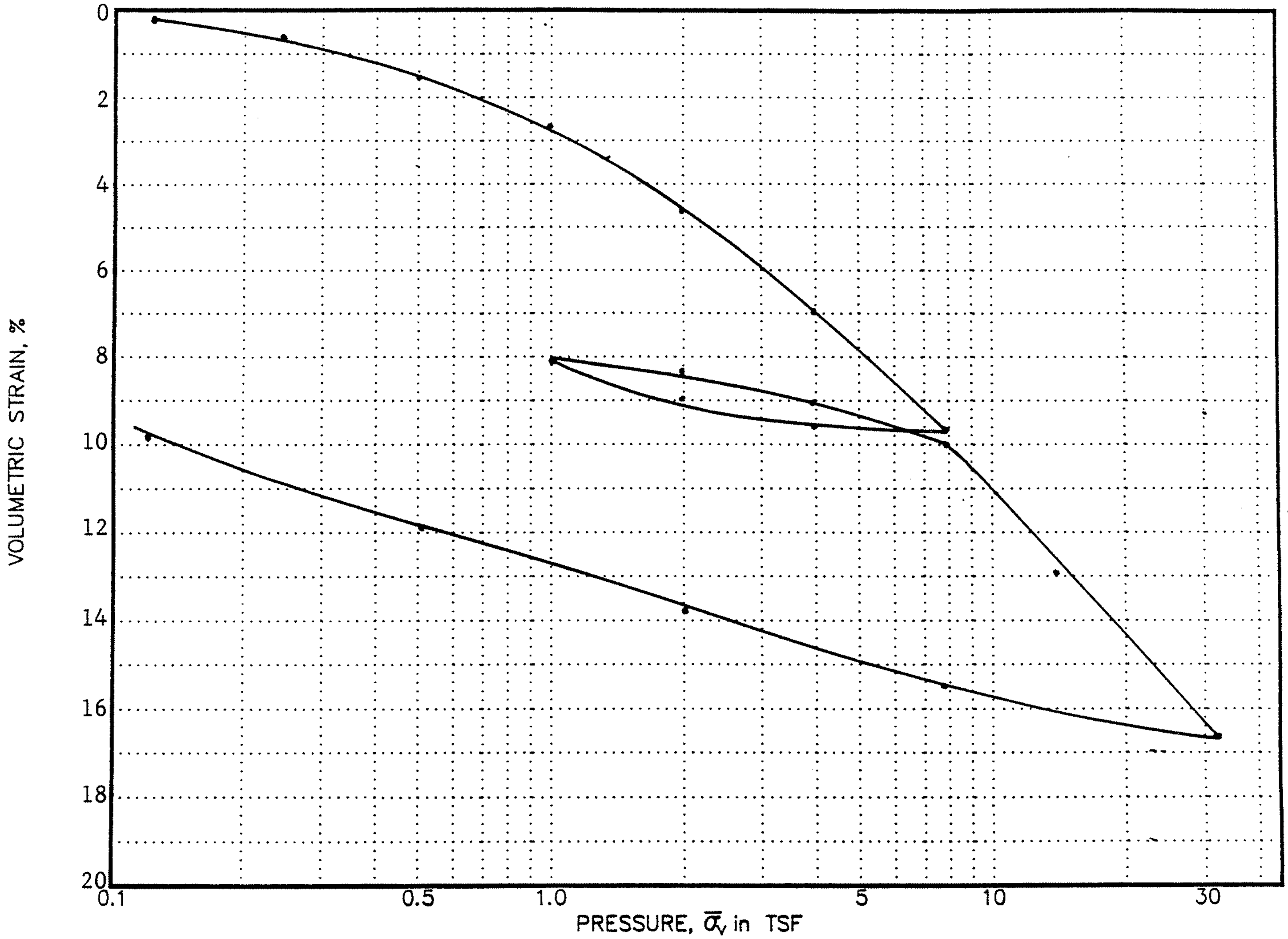


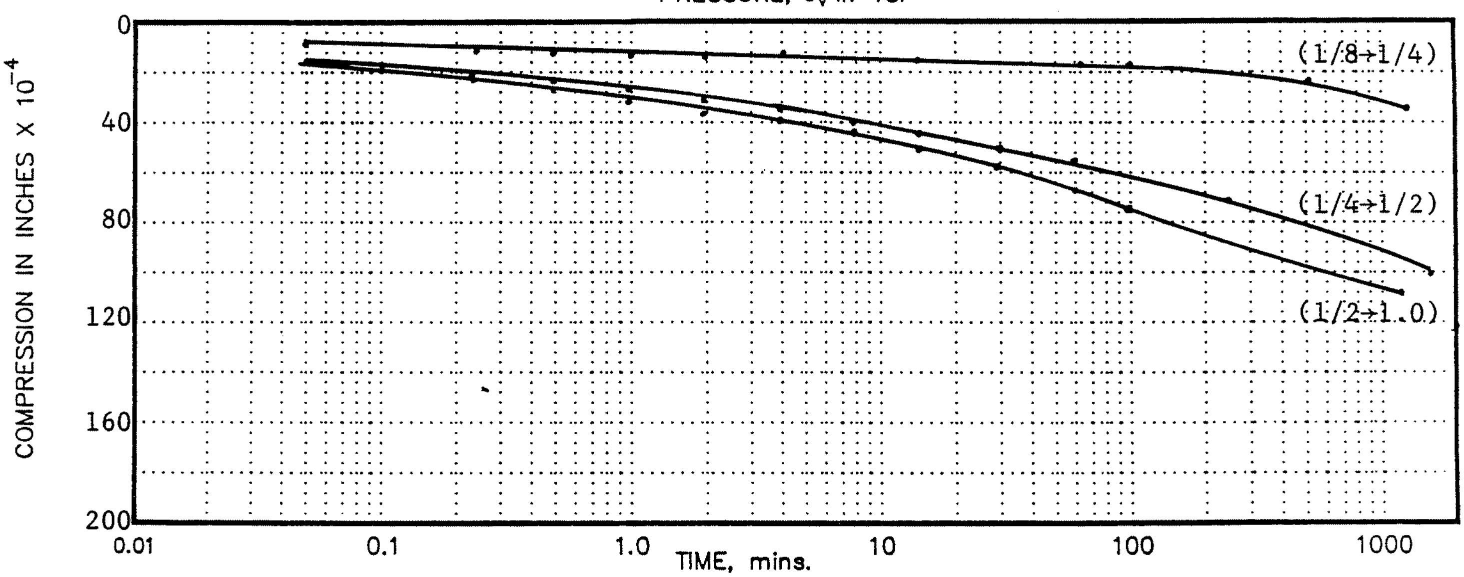
CONSOLIDATION TEST

JOB NO.: 89C 2857 JOB NAME: Olin-Gibson

BORING NO.: WCC-5			SAMPLE NO.: ST-1				DEPTH, FT.: 2.35		
MATERIAL: Brown Silty Clay									
	WATER CONTENT,%	VOID RATIO	SATURATION, %	HEIGHT, inches	DRY UNIT WEIGHT,pcf	DIAMETER, inches	SPECIFIC GRAVITY	LIQUID LIMIT, %	PLASTIC LIMIT, %
INITIAL	26.5	0.715	99.3	0.866					
FINAL	23.6	0.630	100.3	0.823	97.5	2.495	2.68	40	19

COMPRESSION RATIO*	0.113	PRECONSOLIDATION STRESS, TSF	1.8
RECOMPRESSION RATIO*	0.019	EXISTING OVERBURDEN STRESS, TSF	0.15
SWELLING RATIO*	0.030	*FROM VOLUMETRIC STRAIN	



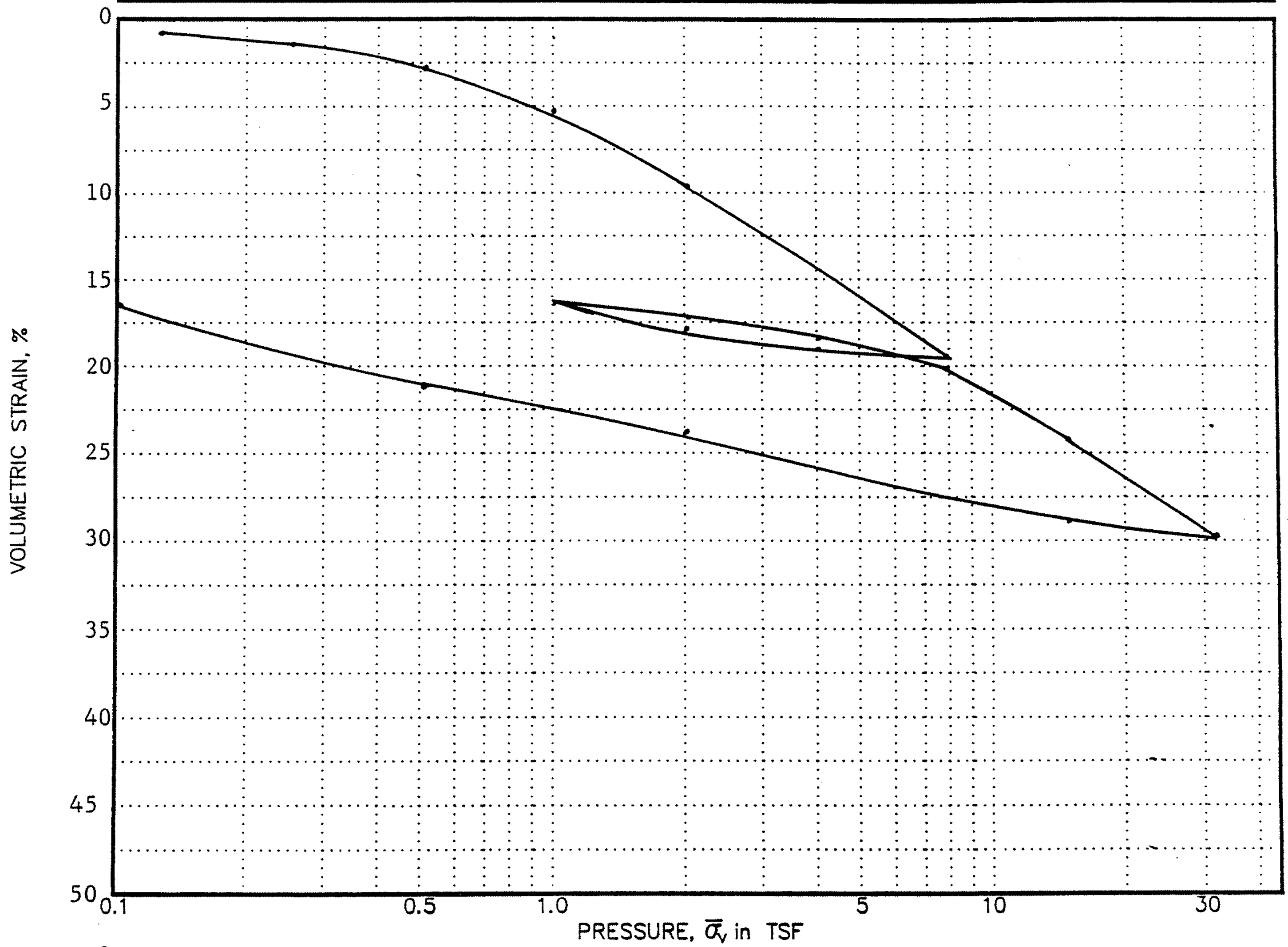


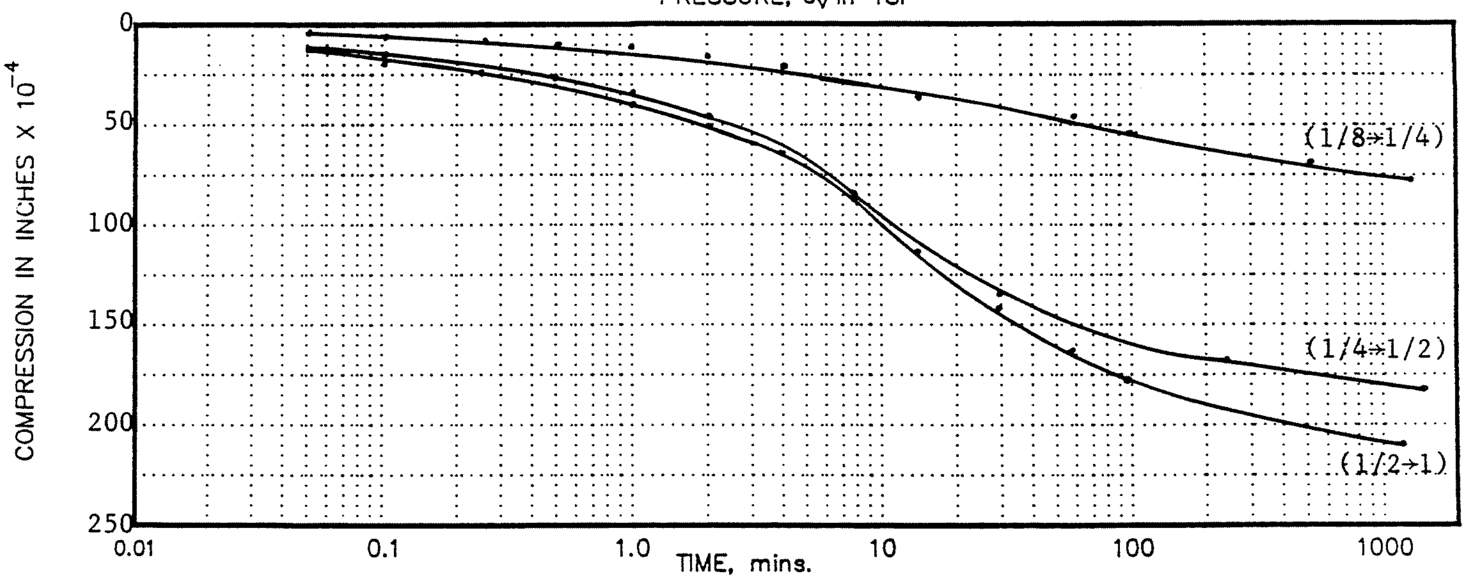
CONSOLIDATION TEST

JOB NO.: 89 C 2857 JOB NAME: Olin-Gibson

BORING NO.: WCC-5				SAMPLE NO.: ST-2		DEPTH, FT.: 5.6		
MATERIAL: Brown silty clay								
	WATER CONTENT, %	VOID RATIO	SATURATION, %	HEIGHT, inches	DRY UNIT WEIGHT, pcf	DIAMETER, inches	SPECIFIC GRAVITY	LIQUID LIMIT, %	PLASTIC LIMIT, %
INITIAL	49.6	1.349	100	0.869	736	2.495	2.77	43	22
FINAL	36.3	0.941	100	0.718					

COMPRESSION RATIO*	0.170	PRECONSOLIDATION STRESS, TSF	1.0
RECOMPRESSION RATIO*	0.030	EXISTING OVERBURDEN STRESS, TSF	0.31
SWELLING RATIO*	0.050	*FROM VOLUMETRIC STRAIN	

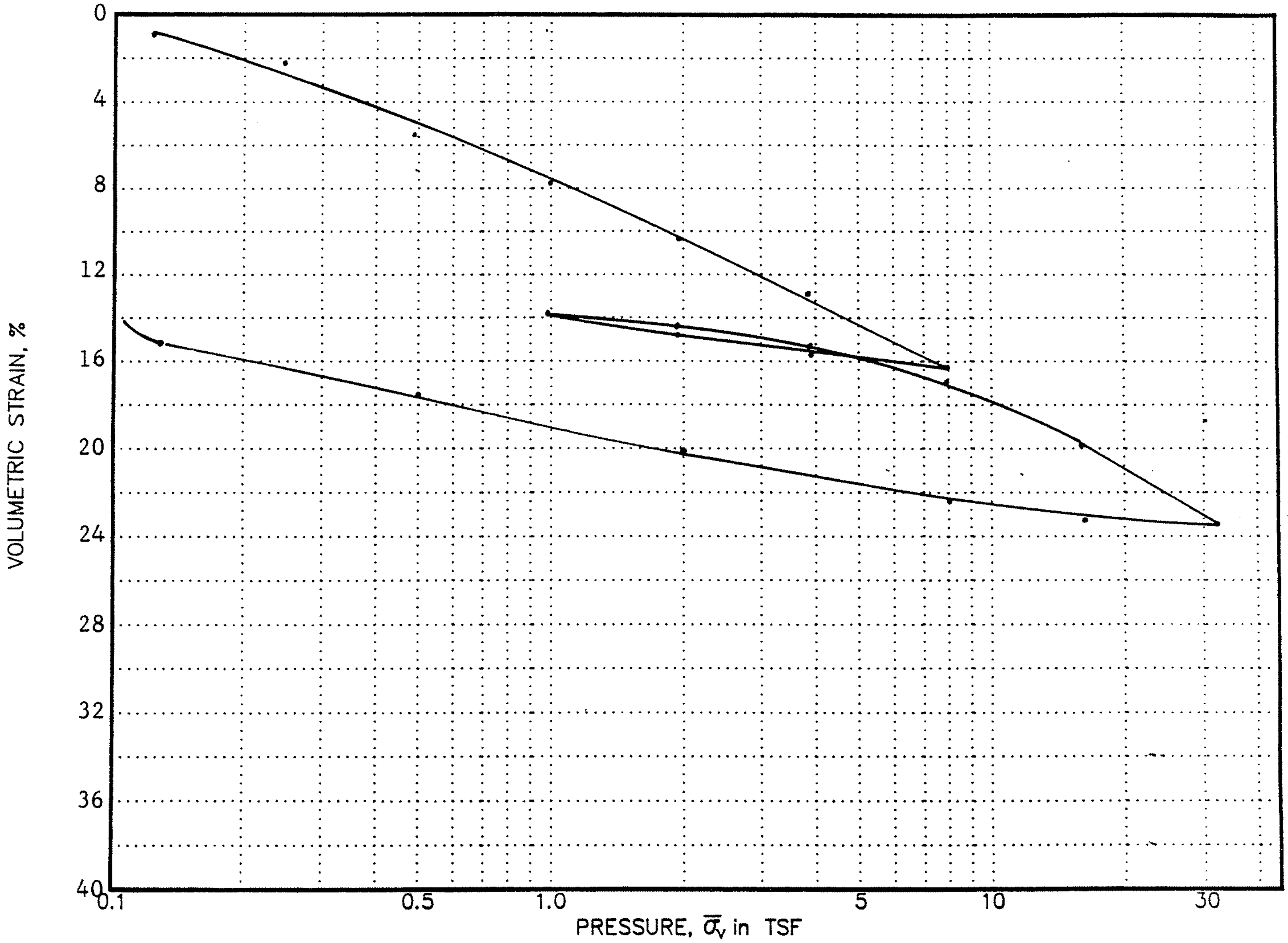


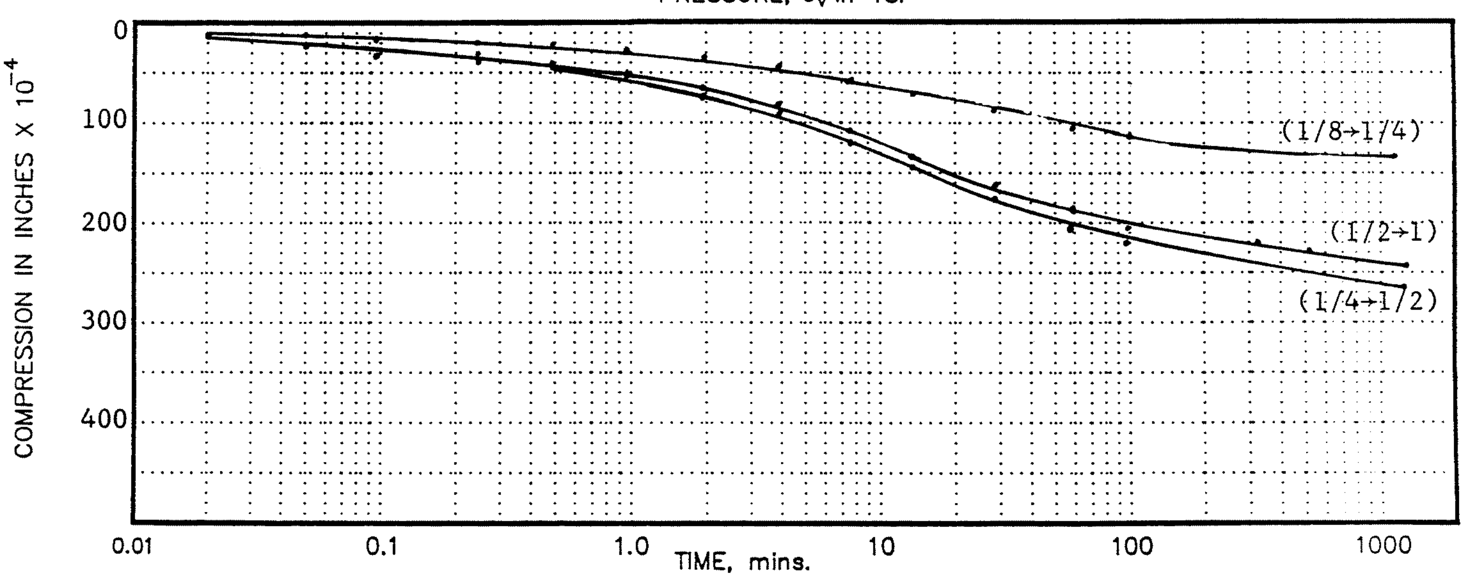


CONSOLIDATION TEST

JOB NO.: 89 C 2857 JOB NAME: Olin-Gibson

BORING NO.: WCC-5			SAMPLE NO.: ST-3			DEPTH, FT.: 8.40			
MATERIAL: Brown silty clay									
	WATER CONTENT, %	VOID RATIO	SATURATION, %	HEIGHT, inches	DRY UNIT WEIGHT, pcf	DIAMETER, inches	SPECIFIC GRAVITY	LIQUID LIMIT, %	PLASTIC LIMIT, %
INITIAL	40.0	1.080	100	0.863	82.5	2.495	2.75	46	24
FINAL	26.9	0.713	100	0.711					
COMPRESSION RATIO*		0.122		PRECONSOLIDATION STRESS, TSF					0.6
RECOMPRESSION RATIO*		0.032		EXISTING OVERBURDEN STRESS, TSF					0.41
SWELLING RATIO*		0.038		*FROM VOLUMETRIC STRAIN					



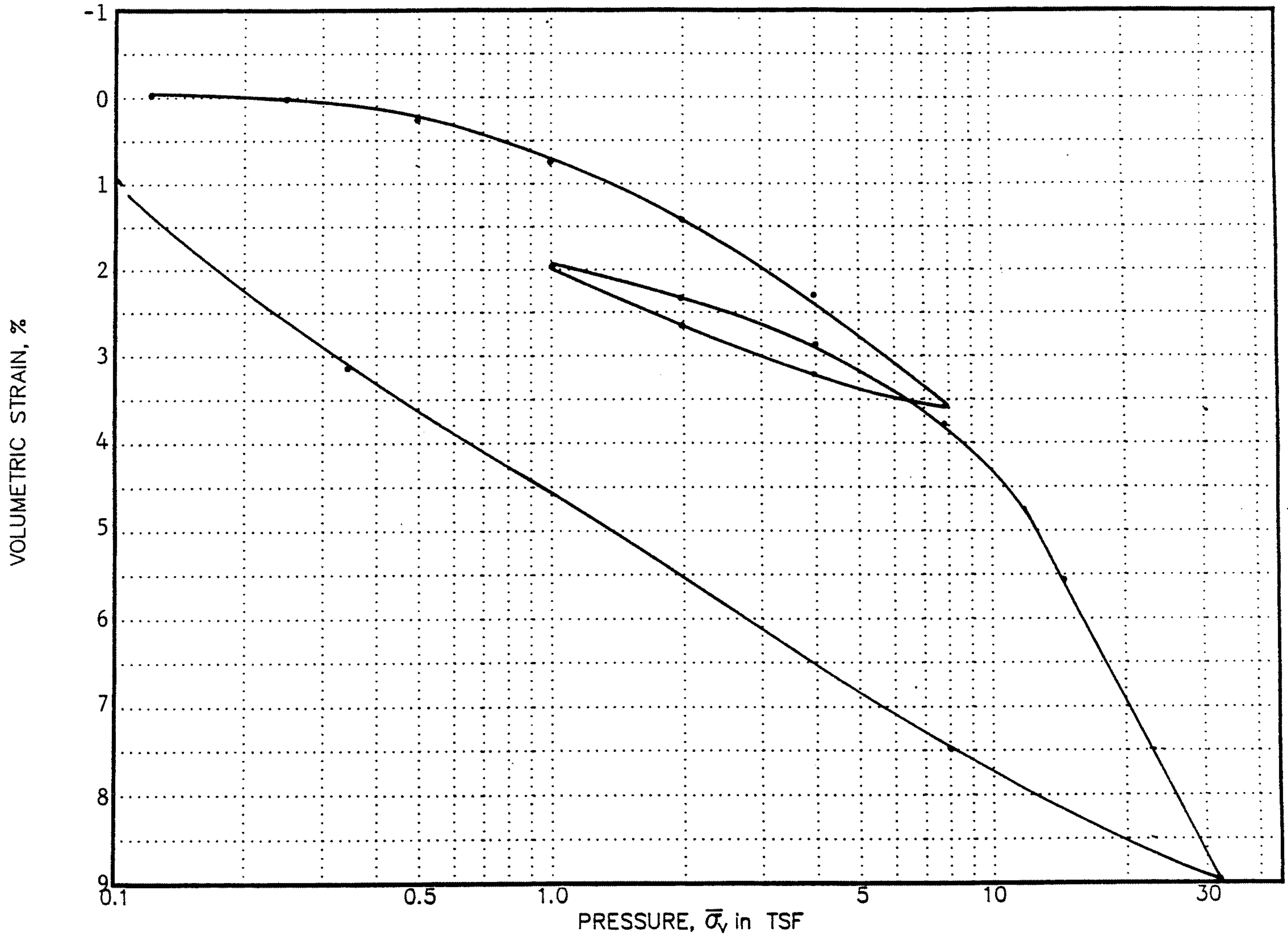


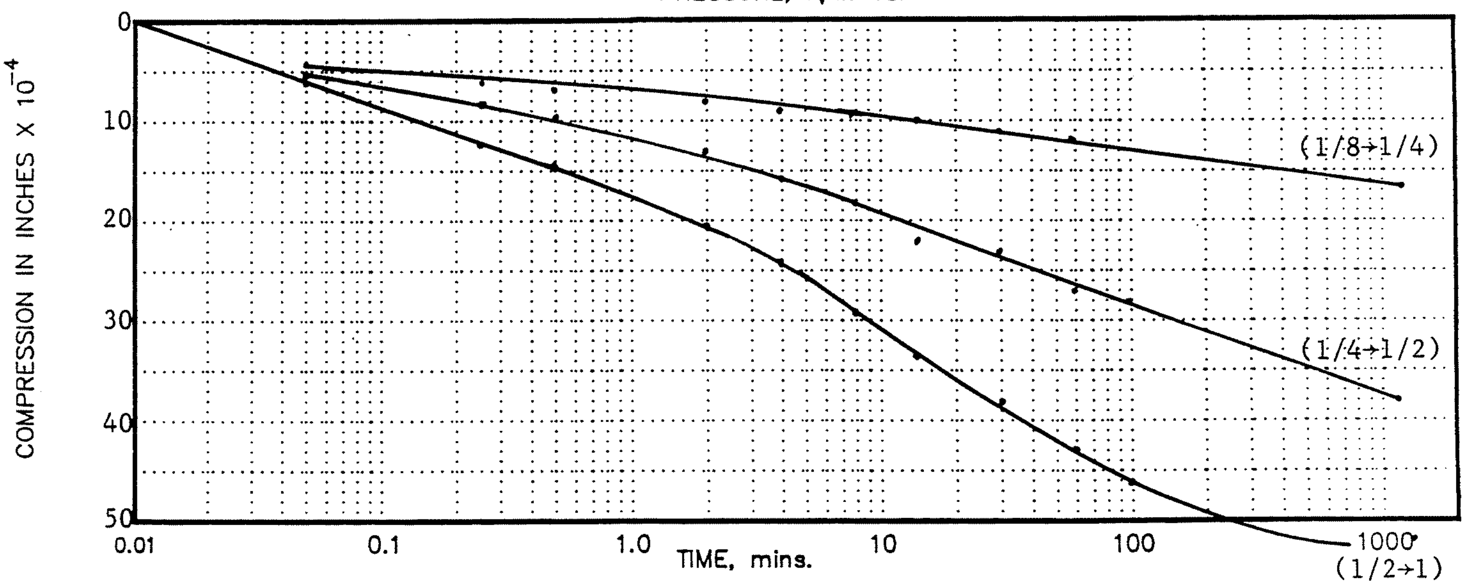
CONSOLIDATION TEST

JOB NO.: 89 C 2857 JOB NAME: Olin-Gibson

BORING NO.: WCC-6			SAMPLE NO.: ST-3			DEPTH, FT.: 8.6			
MATERIAL: Brown Silty Clay									
	WATER CONTENT,%	VOID RATIO	SATURATION, %	HEIGHT, inches	DRY UNIT WEIGHT,pcf	DIAMETER, inches	SPECIFIC GRAVITY	LIQUID LIMIT, %	PLASTIC LIMIT, %
INITIAL	26.1	0.710	100	0.867	100.9	2495	276	49	23
FINAL	27.4	0.722	100	0.873					

COMPRESSION RATIO*	0.097	PRECONSOLIDATION STRESS, TSF	7.0
RECOMPRESSION RATIO*	0.018	EXISTING OVERBURDEN STRESS, TSF	0.55
SWELLING RATIO*	0.032	*FROM VOLUMETRIC STRAIN	



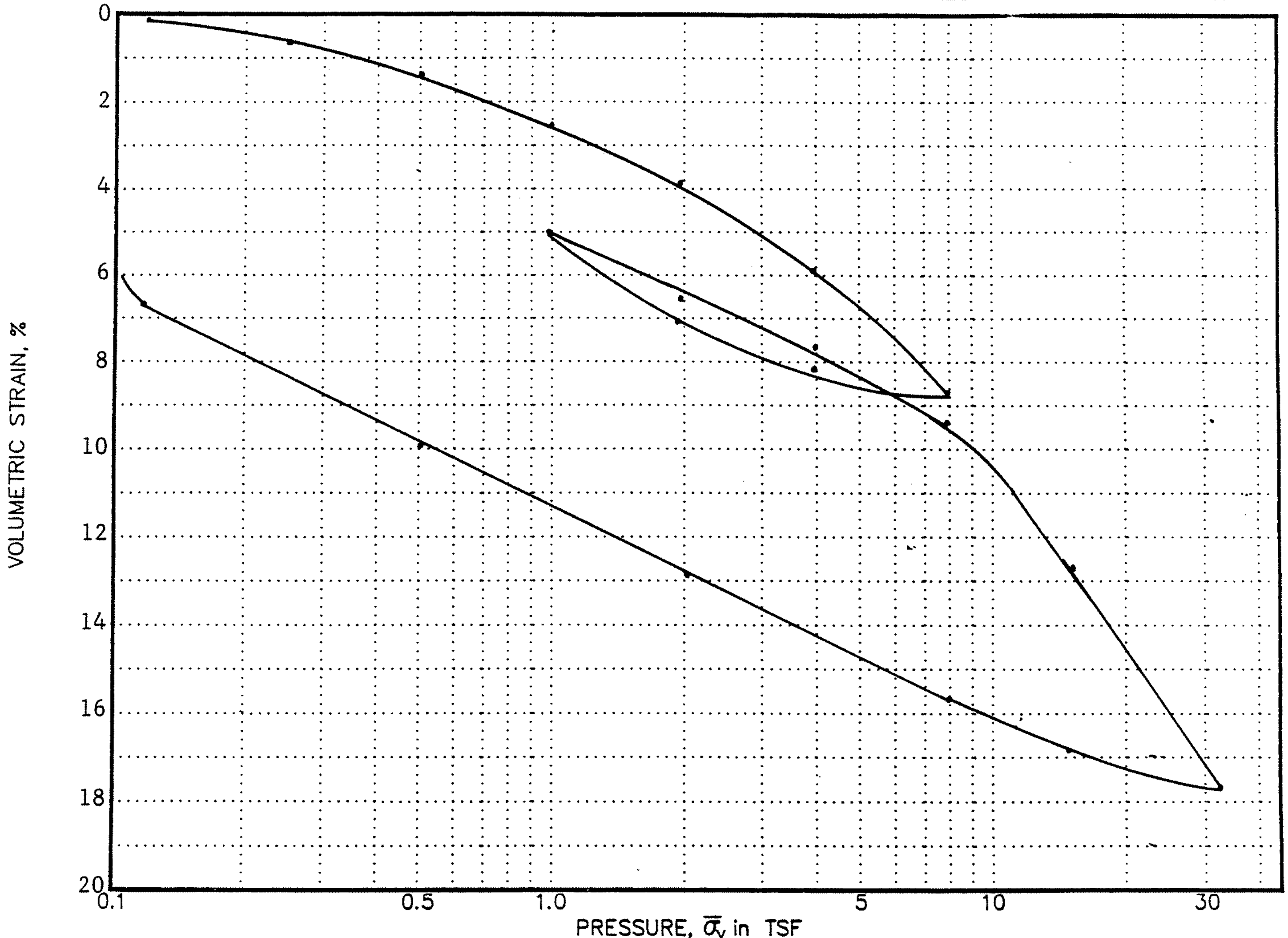


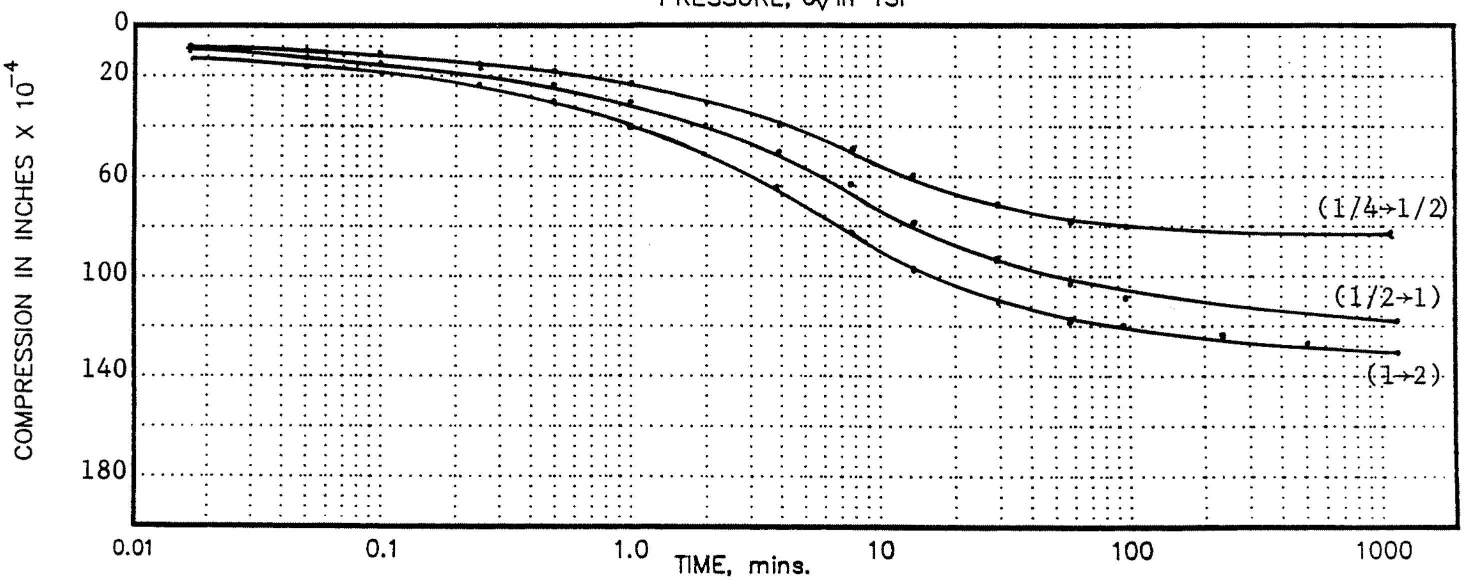
CONSOLIDATION TEST

JOB NO.: 89 C 2857 JOB NAME: Olin-Gibson

BORING NO.: WCC-6			SAMPLE NO.: ST-5			DEPTH, FT.: 13.55		
MATERIAL: Brown silty clay								
	WATER CONTENT,%	VOID RATIO	SATURATION, %	HEIGHT, inches	DRY UNIT WEIGHT,pcf	DIAMETER, inches	SPECIFIC GRAVITY	LIQUID LIMIT, %	PLASTIC LIMIT, %
INITIAL	34.1	0.935	100	0.867	89.6	2.495	2.77	37	20
FINAL	32.8	0.848	100	0.828					

COMPRESSION RATIO*	0.143	PRECONSOLIDATION STRESS, TSF	3.7
RECOMPRESSION RATIO*	0.043	EXISTING OVERBURDEN STRESS, TSF	0.80
SWELLING RATIO*	0.045	*FROM VOLUMETRIC STRAIN	



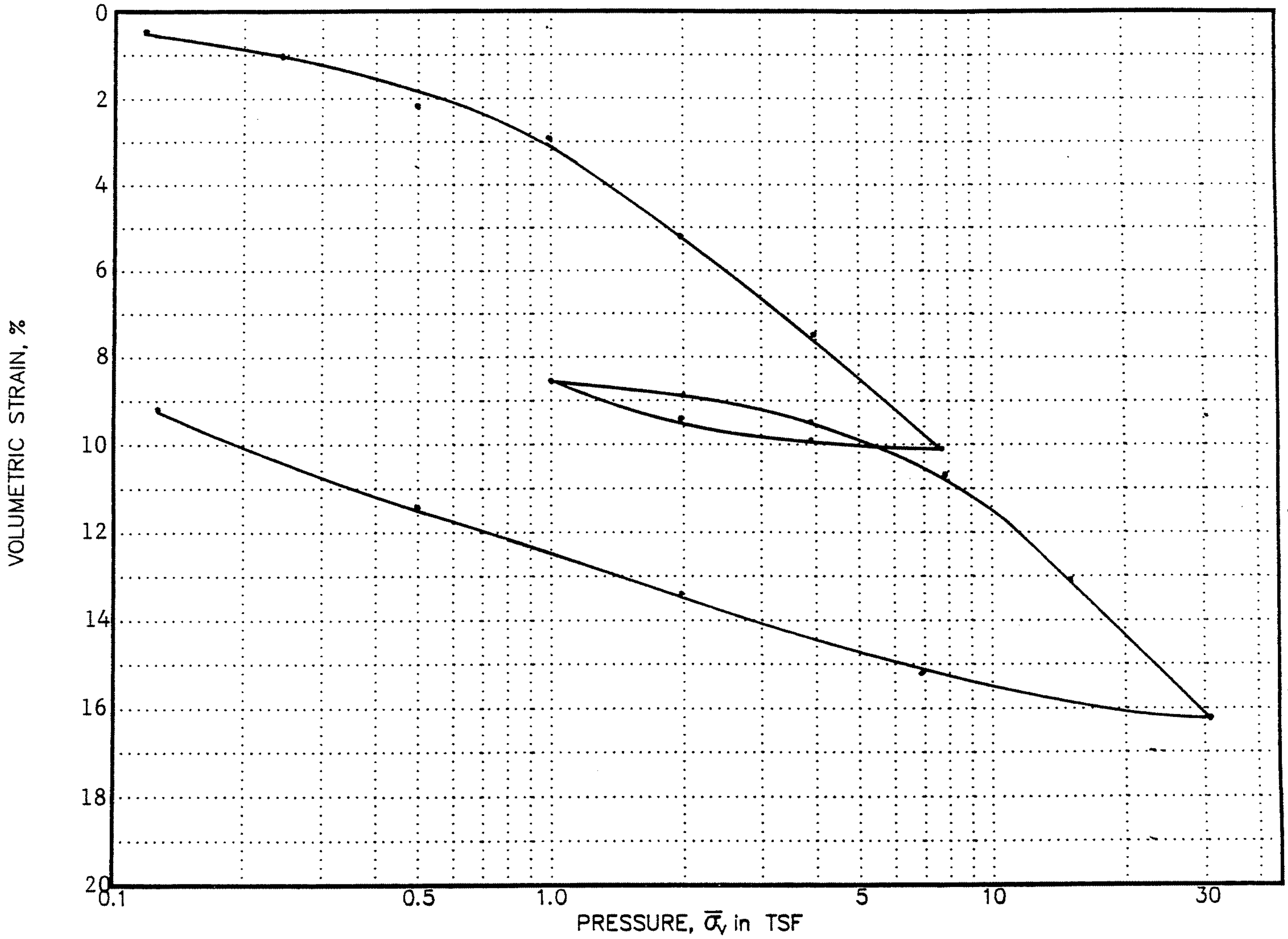


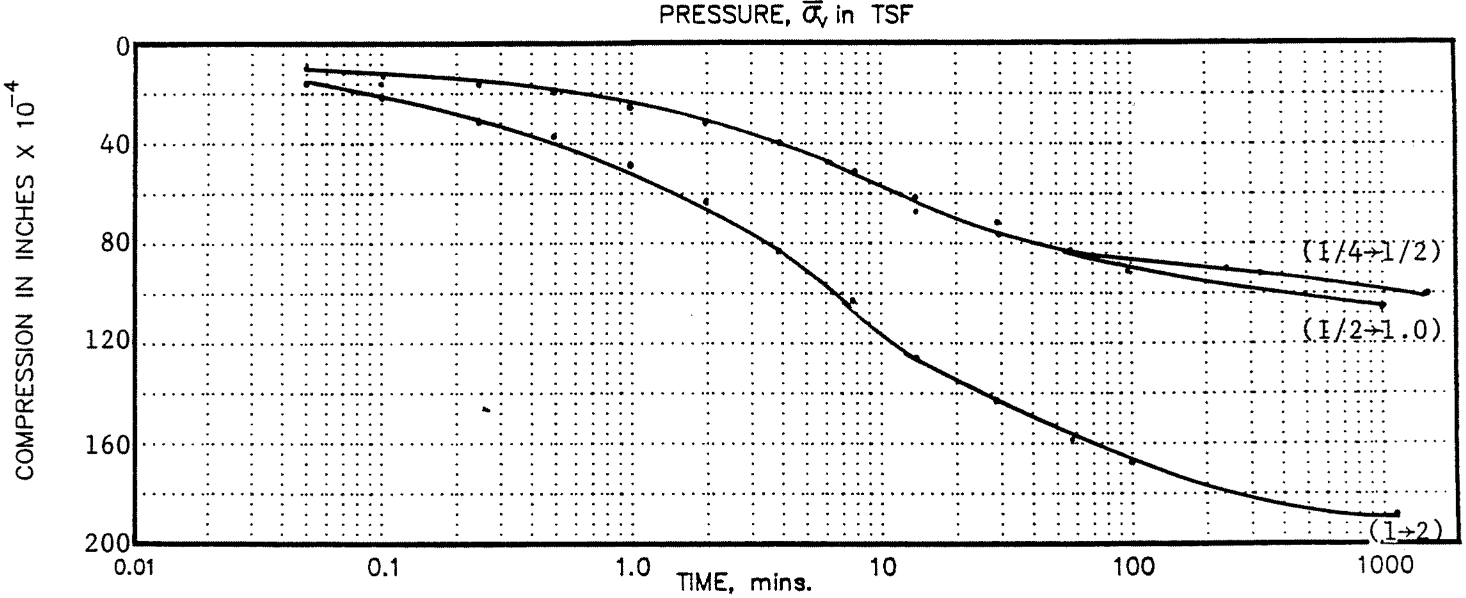
CONSOLIDATION TEST

JOB NO.: 89 C 2857 JOB NAME: Olin-Gibson Site

BORING NO.: WCC-6			SAMPLE NO.: ST-6			DEPTH, FT.: 15.45		
MATERIAL: Brown Silty Clay (CL)								
	WATER CONTENT,%	VOID RATIO	SATURATION, %	HEIGHT, inches	DRY UNIT WEIGHT,pcf	DIAMETER, inches	SPECIFIC GRAVITY	LIQUID LIMIT, %	PLASTIC LIMIT, %
INITIAL	30.7	0.842	100	0.868					
FINAL	25.6	0.692	100	0.797	94.8	2.495	2.75	38	22

COMPRESSION RATIO*	0.100	PRECONSOLIDATION STRESS, TSF	1.9
RECOMPRESSION RATIO*	0.020	EXISTING OVERBURDEN STRESS, TSF	0.86
SWELLING RATIO*	0.033	*FROM VOLUMETRIC STRAIN	

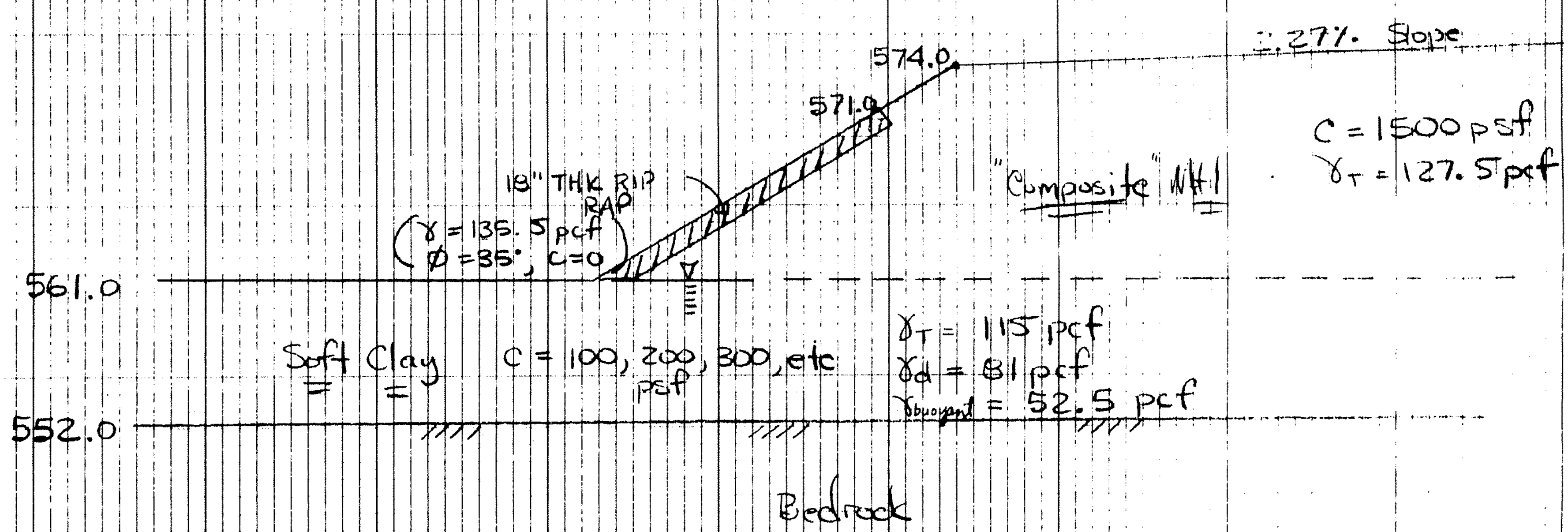




Appendix C

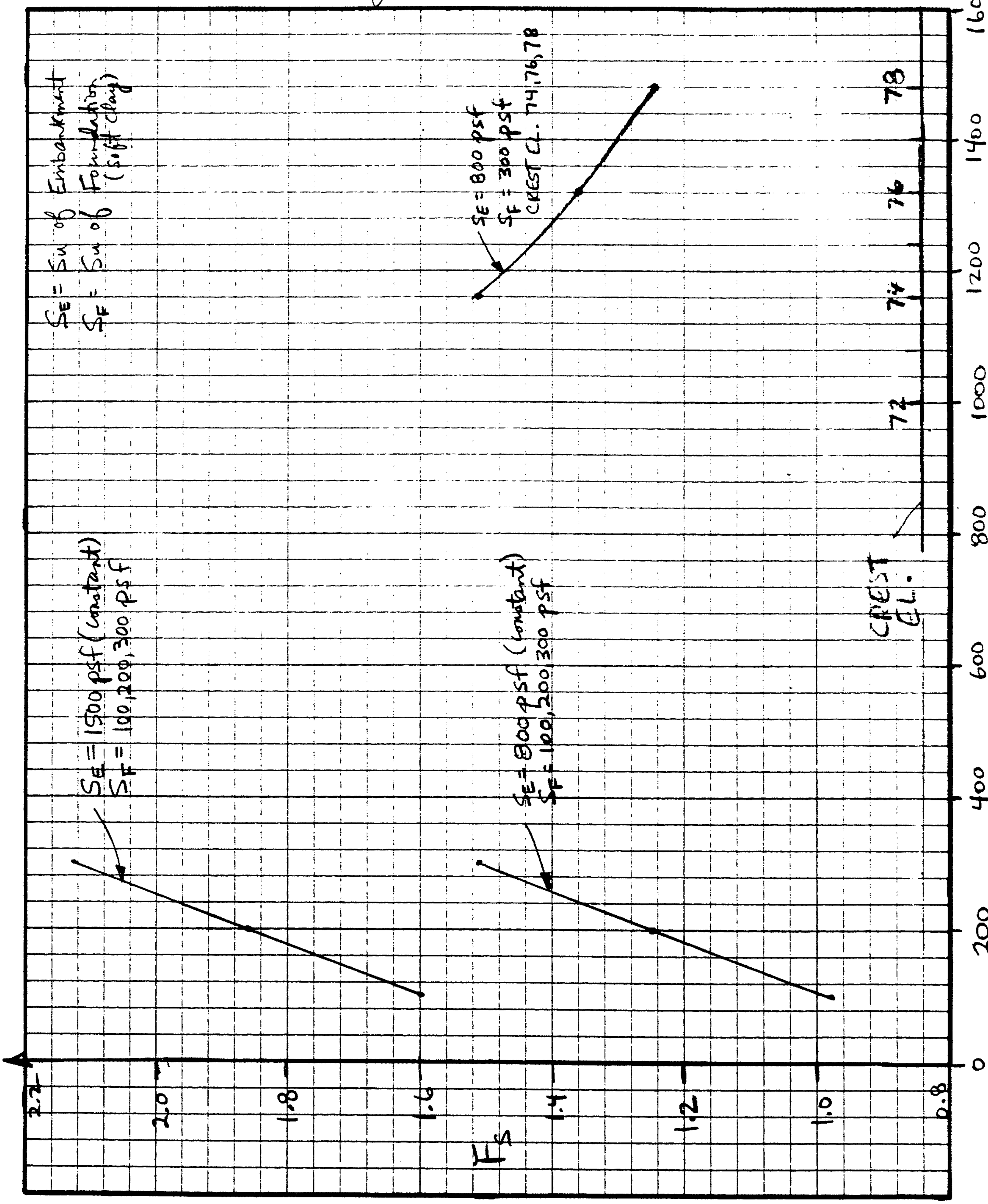
Olin/Gibson Site 89C2857

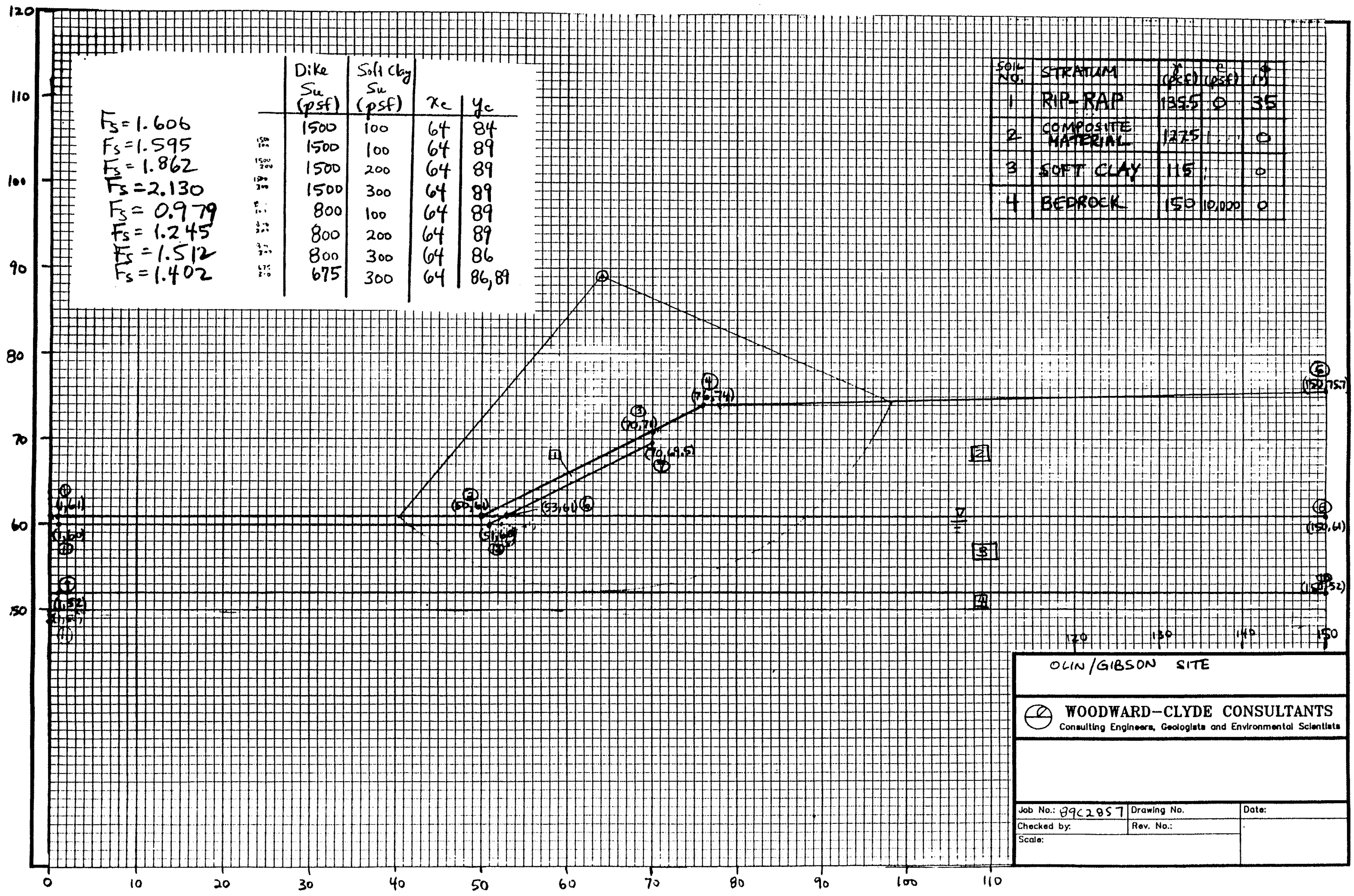
* for Slope Stability Analysis



By: JCV Date: 2/14/90 Subject: Fs vs. Su (for embankment & foundation) Sheet ____ of ____

Chkd. by: JtW Date: 2/22/90 Job No.: 89C2857





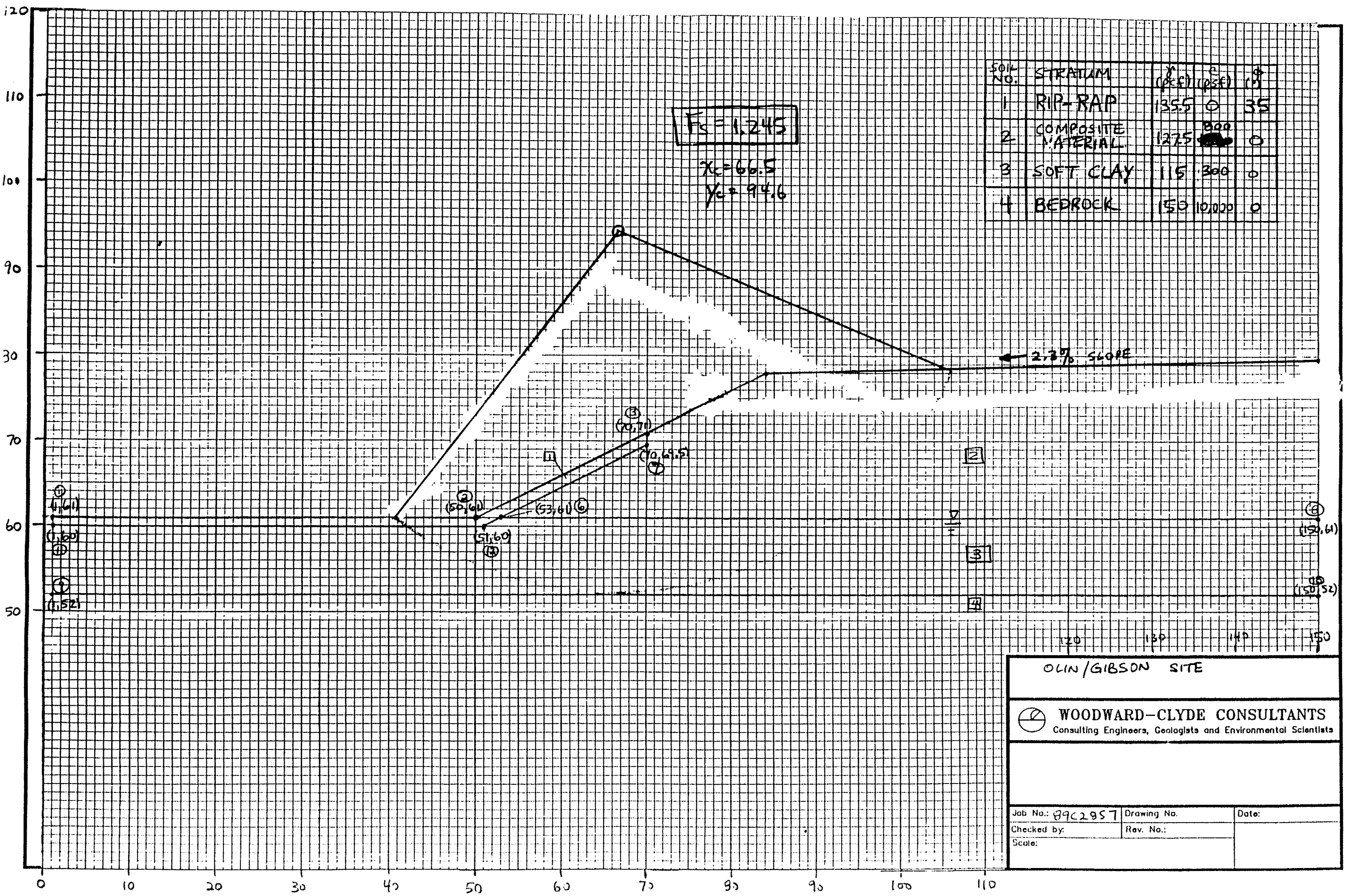
	Dike S_u (psf)	Soft Clay S_u (psf)	x_c	y_c
$F_s = 1.606$	1500	100	64	84
$F_s = 1.595$	1500	100	64	89
$F_s = 1.862$	1500	200	64	89
$F_s = 2.130$	1500	300	64	89
$F_s = 0.979$	800	100	64	89
$F_s = 1.245$	800	200	64	89
$F_s = 1.512$	800	300	64	86
$F_s = 1.402$	675	300	64	86,89

Soil No.	Stratum	γ (pcf)	c (psf)	ϕ (°)
1	RIP-RAP	135.5	0	35
2	COMPOSITE MATERIAL	127.5		0
3	SOFT CLAY	115		0
4	BEDROCK	150	10,000	0

OLIN/GIBSON SITE

WOODWARD-CLYDE CONSULTANTS
Consulting Engineers, Geologists and Environmental Scientists

Job No.: 89C2857	Drawing No.	Date:
Checked by:	Rev. No.:	
Scale:		

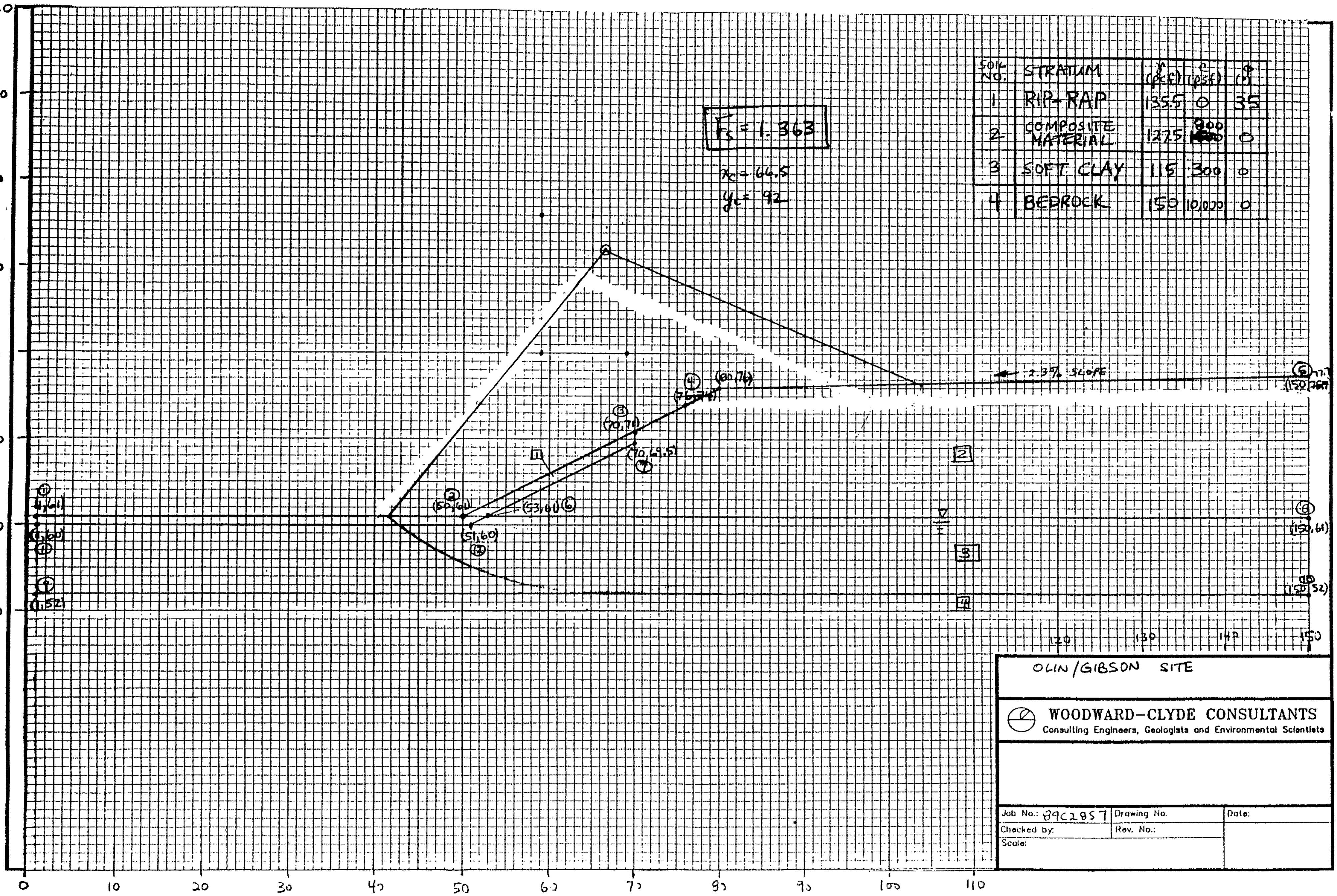


SOIL NO.	STRATUM	γ (pcf)	c (psf)	φ (°)
1	RIP-RAP	135.5	0	35
2	COMPOSITE MATERIAL	127.5	800	0
3	SOFT CLAY	115	300	0
4	BEDROCK	150	10,000	0

OLIN/GIBSON SITE

WOODWARD-CLYDE CONSULTANTS
Consulting Engineers, Geologists and Environmental Scientists

Job No.: 89C2857	Drawing No.	Date:
Checked by:	Rev. No.:	
Scale:		



SOIL NO. | STRATUM | γ (pcf) | c (psf) | φ (°)
1 | RIP-RAP | 135.5 | 0 | 35
2 | COMPOSITE MATERIAL | 127.5 | 800 | 0
3 | SOFT CLAY | 115 | 300 | 0
4 | BEDROCK | 150 | 10,000 | 0
$F_s = 1.363$
$x_c = 66.5$
$y_c = 92$
2.3% SLOPE
(80,76)
(76,74)
(70,71)
(70,69.5)
(50,60)
(53,60)
(51,60)
(1,61)
(1,60)
(1,52)
(150,77)
(150,61)
(150,52)
OLIN/GIBSON SITE
WOODWARD-CLYDE CONSULTANTS
Consulting Engineers, Geologists and Environmental Scientists
Job No.: 89C2857
Drawing No.
Date:
Checked by:
Rev. No.:
Scale:

Appendix D

DESIGN CALCULATIONS

Woodward-Clyde Consultants Consulting Engineers. Geologists and Environmental Scientists	Owner: OLIN	Page ___ of ___
	Project:	Job No./File No. 89C2857
System:		Calculator JCV Date 2/23/90
Calculation for: SETTLEMENT CALCULATIONS		Reviewer [illegible] Date 2/23/90

SUMMARY OF SETTLEMENT RESULTS

CASE	PRECONSOLIDATION PRESSURE OF SOFT CLAY (PSF)	TOTAL SETTLEMENT (INCHES)
1	1200	4.5
2	1800	3.0
3	1200	1.9
4	1800	1.1

NOTE: SEE COMPUTER OUTPUT FOR SKETCHES

DESIGN CALCULATIONS

Woodward-Clyde Consultants Consulting Engineers, Geologists and Environmental Scientists	Owner: OLIN	Page ___ of ___
	Project:	Job No./File No. 89C2857
System:		Calculator JCV Date 2/23/90
Calculation for: SETTLEMENT CALCULATIONS		Reviewer [illegible] Date 2/23/90

DESCRIPTION OF DATA INPUT:

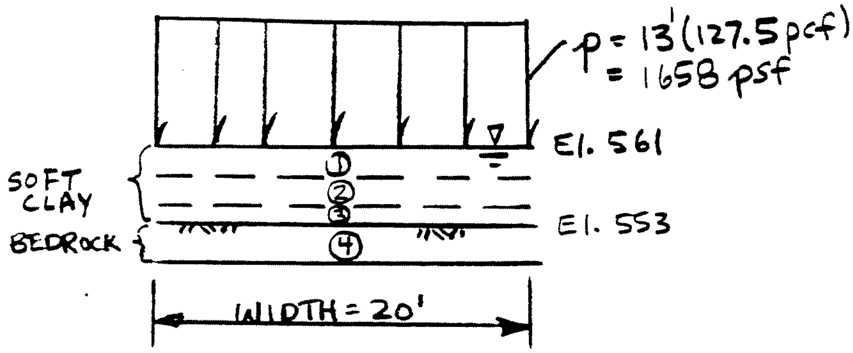
SOIL:

TOP- TOP ELEVATION OF SOIL LAYER (FEET)
BOTTOM- BOTTOM ELEVATION OF SOIL LAYER (FEET)
GAMMA- EFFECTIVE UNIT WEIGHT OF THE SOIL LAYER (PCF)
CC- VIRGIN COMPRESSION INDEX (UNIT STRAIN BASIS) OF THE SOIL LAYER.
CR- RECOMPRESSION INDEX (UNIT STRAIN BASIS) OF THE SOIL LAYER
PC- PRECONSOLIDATION PRESSURE OF THE SOIL LAYER (PSF)
E- DEFORMATION MODULUS OF THE SOIL LAYER ($PSF \times 10^6$)
MU- POISSON'S RATIO OF THE SOIL LAYER
CA- COEFFICIENT OF SECONDARY COMPRESSION (UNIT STRAIN BASIS) OF THE SOIL LAYER
A- PORE PRESSURE COEFFICIENT OF THE SOIL LAYER
TZERO- TIME REQUIRED FOR PRIMARY CONSOLIDATION OF THE SOIL LAYER (YEARS)

LOAD:

CX- THE X-COORDINATE OF THE CENTER OF A PARTICULAR RECTANGULAR LOADED AREA (FEET)
CY- THE Y-COORDINATE OF THE CENTER OF THE LOADED AREA (FEET)
ELEV- THE BASE ELEVATION OF THE LOADED AREA (FEET)
WIDTH- THE WIDTH OF THE LOADED AREA IN THE X-DIRECTION (FEET)
LENGTH- THE LENGTH OF THE LOADED AREA IN THE Y-DIRECTION (FEET)
LOAD – THE AVERAGE CONTACT PRESSURE OF THE LOADED AREA (PSF)

CASE I



EFORM.FOR - VERSION 1.1, MARCH 1981

FOR SETTLEMENT OPTION INPUT 1
OR STRESS DISTRIBUTION ONLY INPUT 2

1
F DATA ON DISK FILE TYPE 1
F DATA AT TERMINAL TYPE 2

YPE FILE NAME:

A:D1.DAT
UMBER OF SOIL LAYERS: 4

SOIL	TOP	BOTTOM	GAMMA	CC	CR	PC	E*10**6	MU	CA	A	TZERO
1	561.0	558.0	54.0	.120	.030	1200.0	9999.000	.50	.00000	1.000	1.00
2	558.0	555.0	54.0	.120	.030	1200.0	9999.000	.50	.00000	1.000	1.00
3	555.0	553.0	54.0	.120	.030	1200.0	9999.000	.50	.00000	1.000	1.00
4	553.0	550.0	90.0	.000	.000	10000.0	9999.000	.40	.00000	1.000	1.00

PROJECT LIFE: 30.0

UMBER OF LOADED AREAS: 1

REA	CX	CY	ELEV	WIDTH	LENGTH	LOAD
1	.00	.00	561.00	20.00	100.00	1658.00

TO BEGIN EXECUTION, TYPE 1
O PRINT INPUT DATA, TYPE 2
O CHANGE SOIL DATA, TYPE 3
TO CHANGE LOAD DATA, TYPE 4
O STOP, TYPE 5

1

IPUT LOCATION OF SETTLEMENT POINT - X,Y:
0,0

OIL LAYER	INITIAL STRESS (PSF)	VERTICAL STRESS INCREASE (PSF)	HORIZ. X STRESS INCREASE (PSF)	HORIZ. Y STRESS INCREASE (PSF)	SKEMPTON BJERRUM FACTOR	CONSOL. COMP. (IN.)	SECOND. COMP. 30. YR (IN.)	ELASTIC COMP. (IN.)	TOTAL COMP. (IN.)
1	81.0	1655.7	1345.9	1491.5	1.000	1.958	.000	.000	1.958
2	243.0	1606.6	816.5	1183.9	1.000	1.561	.000	.000	1.561
3	378.0	1508.8	517.2	970.8	1.000	.927	.000	.000	.927
4	567.0	1382.1	328.6	799.4	1.000	.000	.000	.000	.000
						-------	-------	-------	-------
						4.446	.000	.000	4.446

TO BEGIN EXECUTION, TYPE 1
O PRINT INPUT DATA, TYPE 2

CASE 2 (see CASE 1 for sketch)


```
~EFORM.FOR - VERSION 1.1, MARCH 1981


FOR SETTLEMENT OPTION INPUT 1
 )R STRESS DISTRIBUTION ONLY INPUT 2

1
  : DATA ON DISK FILE TYPE 1
  : DATA AT TERMINAL TYPE 2

1
 'PE FILE NAME:

A:D1A.DAT
 IMBER OF SOIL LAYERS:   4

SOIL     TOP  BOTTOM  GAMMA    CC     CR       PC  E*10**6    MU      CA      A  TZERO
   1   561.0   558.0   54.0  .120   .030   1800.0 9999.000   .50  .00000 1.000   1.00
   2   558.0   555.0   54.0  .120   .030   1800.0 9999.000   .50  .00000 1.000   1.00
   3   555.0   553.0   54.0  .120   .030   1800.0 9999.000   .50  .00000 1.000   1.00
   4   553.0   550.0   90.0  .000   .000  10000.0 9999.000   .40  .00000 1.000   1.00

..OJECT LIFE:   30.0

 IMBER OF LOADED AREAS:   1


^REA      CX        CY     ELEV    WIDTH   LENGTH     LOAD
   1      .00       .00   561.00    20.00   100.00  1658.00

TO BEGIN EXECUTION, TYPE 1
   PRINT INPUT DATA, TYPE 2
   CHANGE SOIL DATA, TYPE 3
TO CHANGE LOAD DATA, TYPE 4
   STOP, TYPE 5

1

  PUT LOCATION OF SETTLEMENT POINT - X,Y:
u,0




               VERTICAL  HORIZ. X  HORIZ. Y  SKEMPTON            SECOND.
  IL   INITIAL  STRESS    STRESS    STRESS    BJERRUM   CONSOL.  COMP.    ELASTIC    TOTAL
LAYER  STRESS   INCREASE  INCREASE  INCREASE  FACTOR    COMP.     30. YR  COMP.      COMP.
       (PSF)    (PSF)     (PSF)     (PSF)               (IN.)    (IN.)    (IN.)      (IN.)

  1       81.0    1655.7    1345.9    1491.5     1.000    1.438     .000      .000    1.438
  2      243.0    1606.6     816.5    1183.9     1.000     .990     .000      .000     .990
  3      378.0    1508.8     517.2     970.8     1.000     .547     .000      .000     .547
  4      567.0    1382.1     328.6     799.4     1.000     .000     .000      .000     .000
                                                          -------  -------  -------  -------
                                                          2.975     .000      .000    2.975

TO BEGIN EXECUTION, TYPE 1
TO PRINT INPUT DATA, TYPE 2
```

CASE 3

A:D2.DAT

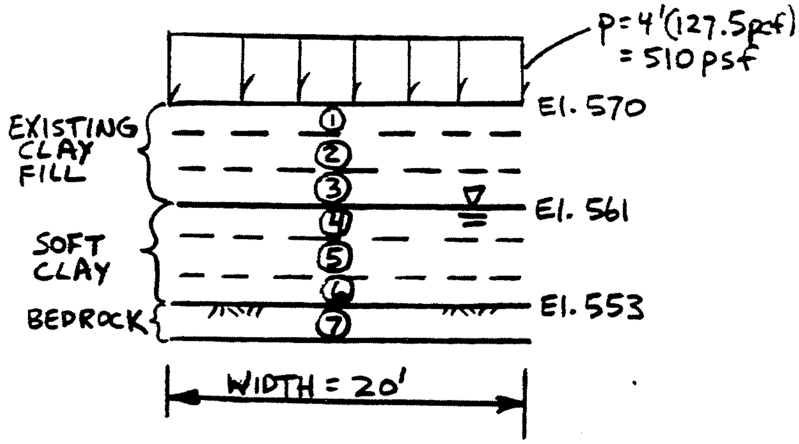
NUMBER OF SOIL LAYERS: 7

OIL	TOP	BOTTOM	GAMMA	CC	CR	PC	E*10**6	MU	CA	A	TZERO
1	570.0	567.0	127.5	.110	.020	3600.0	9999.000	.50	.00000	1.000	1.00
2	567.0	564.0	127.5	.110	.020	3600.0	9999.000	.50	.00000	1.000	1.00
3	564.0	561.0	127.5	.110	.020	3600.0	9999.000	.50	.00000	1.000	1.00
4	561.0	558.0	54.0	.120	.030	1200.0	9999.000	.50	.00000	1.000	1.00
5	558.0	555.0	54.0	.120	.030	1200.0	9999.000	.50	.00000	1.000	1.00
6	555.0	553.0	54.0	.120	.030	1200.0	9999.000	.50	.00000	1.000	1.00
7	553.0	550.0	90.0	.000	.000	10000.0	9999.000	.40	.00000	1.000	1.00

PROJECT LIFE: 30.0

NUMBER OF LOADED AREAS: 1

REA	CX	CY	ELEV	WIDTH	LENGTH	LOAD
1	.00	.00	570.00	20.00	100.00	510.00



) BEGIN EXECUTION, TYPE 1
TO PRINT INPUT DATA, TYPE 2
TO CHANGE SOIL DATA, TYPE 3
) CHANGE LOAD DATA, TYPE 4
.) STOP, TYPE 5

INPUT LOCATION OF SETTLEMENT POINT - X,Y:
0,0

SOIL LAYER	INITIAL STRESS (PSF)	VERTICAL STRESS INCREASE (PSF)	HORIZ. X STRESS INCREASE (PSF)	HORIZ. Y STRESS INCREASE (PSF)	SKEMPTON BJERRUM FACTOR	CONSOL. COMP. (IN.)	SECOND. COMP. 30. YR (IN.)	ELASTIC COMP. (IN.)	TOTAL COMP. (IN.)
1	191.3	509.3	414.0	458.8	1.000	.406	.000	.000	.406
2	573.8	494.2	251.2	364.2	1.000	.194	.000	.000	.194
3	956.3	456.8	145.1	287.1	1.000	.122	.000	.000	.122
4	1228.5	408.8	84.8	228.0	1.000	.539	.000	.000	.539
5	1390.5	361.5	51.6	183.3	1.000	.434	.000	.000	.434
6	1525.5	326.0	35.3	154.2	1.000	.242	.000	.000	.242
7	1714.5	294.8	24.9	130.6	1.000	.000	.000	.000	.000
						-------	-------	-------	-------
						1.937	.000	.000	1.937

TO BEGIN EXECUTION, TYPE 1
I PRINT INPUT DATA, TYPE 2
IO CHANGE SOIL DATA, TYPE 3
TO CHANGE LOAD DATA, TYPE 4
I STOP, TYPE 5

5

Stop - Program terminated.

C:\USR\DEFORM>

CASE 4 (see CASE 3 for sketch)

A:D2A.DAT

NUMBER OF SOIL LAYERS: 7

OIL	TOP	BOTTOM	GAMMA	CC	CR	PC	E*10**6	MU	CA	A	TZERO
1	570.0	567.0	127.5	.110	.020	3600.0	9999.000	.50	.00000	1.000	1.00
2	567.0	564.0	127.5	.110	.020	3600.0	9999.000	.50	.00000	1.000	1.00
3	564.0	561.0	127.5	.110	.020	3600.0	9999.000	.50	.00000	1.000	1.00
4	561.0	558.0	54.0	.120	.030	1800.0	9999.000	.50	.00000	1.000	1.00
5	558.0	555.0	54.0	.120	.030	1800.0	9999.000	.50	.00000	1.000	1.00
6	555.0	553.0	54.0	.120	.030	1800.0	9999.000	.50	.00000	1.000	1.00
7	553.0	550.0	90.0	.000	.000	10000.0	9999.000	.40	.00000	1.000	1.00

ROJECT LIFE: 30.0

NUMBER OF LOADED AREAS: 1

AREA	CX	CY	ELEV	WIDTH	LENGTH	LOAD
1	.00	.00	570.00	20.00	100.00	510.00

) BEGIN EXECUTION, TYPE 1
TO PRINT INPUT DATA, TYPE 2
TO CHANGE SOIL DATA, TYPE 3
) CHANGE LOAD DATA, TYPE 4
IO STOP, TYPE 5

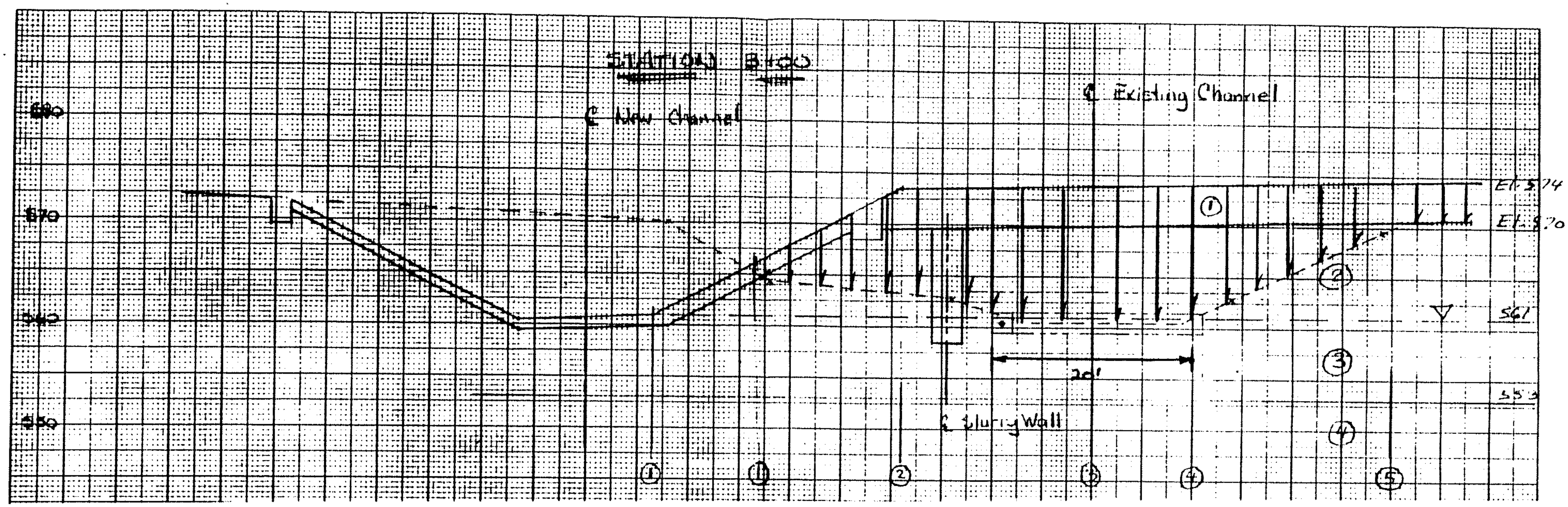
INPUT LOCATION OF SETTLEMENT POINT - X,Y:
,0

SOIL LAYER	INITIAL STRESS (PSF)	VERTICAL STRESS INCREASE (PSF)	HORIZ. X STRESS INCREASE (PSF)	HORIZ. Y STRESS INCREASE (PSF)	SKEMPTON BJERRUM FACTOR	CONSOL. COMP. (IN.)	SECOND. COMP. 30. YR (IN.)	ELASTIC COMP. (IN.)	TOTAL COMP. (IN.)
1	191.3	509.3	414.0	458.8	1.000	.406	.000	.000	.406
2	573.8	494.2	251.2	364.2	1.000	.194	.000	.000	.194
3	956.3	456.8	145.1	287.1	1.000	.122	.000	.000	.122
4	1228.5	408.8	84.8	228.0	1.000	.135	.000	.000	.135
5	1390.5	361.5	51.6	183.3	1.000	.108	.000	.000	.108
6	1525.5	326.0	35.3	154.2	1.000	.087	.000	.000	.087
7	1714.5	294.8	24.9	130.6	1.000	.000	.000	.000	.000
						-------	-------	-------	-------
						1.052	.000	.000	1.052

TO BEGIN EXECUTION, TYPE 1
PRINT INPUT DATA, TYPE 2
TO CHANGE SOIL DATA, TYPE 3
TO CHANGE LOAD DATA, TYPE 4
STOP, TYPE 5

5
op - Program terminated.

C:\USR\DEFORM>



① Cap – Assume 4' @ 127.5 psf = 510 psf

② Existing or New Fill. Assume 127.5 psf
(Max. Fill Loading = 9 + 4 = 13' × 127.5 = 1,657.5)
p_c = 3600 psf
C_r' = 0.019
C_c' = 0.11

③ Very Soft Lacustrine Clay
γ_t = 116.4 pcf
γ_b = 54.0 pcf
C_c' = 0.12
C_r' = 0.03
P_c = 1,800 psf – Case ①
P_c = 1,200 – Case ②

④ Incompressible Soil or Rock

Appendix E

Memorandum **Woodward-Clyde Consultants**

To: Frank S. Waller

From: John B. Stevens

Office: Plymouth Meeting

Date: February 7, 1990

Subject: Rip-Rap Design for Gibson Site

I have completed the tasks given in your memorandum of February 2, 1990. Attached is the cover sheet for the rip-rap design calculations describing step-by-step the procedure used and reasons for selecting various criteria, etc.

I also performed the rip-rap design procedure in reverse to determine the conditions under which "medium" rip-rap should be used. The COE method and a design chart both come up with a velocity of 15 fps as the limiting velocity for quarried rip-rap. For "rounded cobbles" such as over-size material from a sand pit, the design chart recommends a limiting velocity of 10 fps. Since the mean velocity for a 500-year flood is only 6.7 fps and quarried rip-rap will be used, I believe that the "light" rip-rap will be adequate.

I plotted the gradation of the rip-rap used to line nearby creeks with that for medium and light rip-rap. The rip-rap used has a gradation finer than either the medium or light and corresponds to fine rip-rap specification. Thus light rip-rap at the Gibson site will be more erosion and scour resistant than what the State has used elsewhere.

GENERAL PROCEDURE FOR RIP-RAP DESIGN

There are several procedures for the design of rip-rap protection for channels. Each method is empirically based on field observations and experimental modeling. These procedures fall into two groups; one group bases its design on mean channel velocity and the other on boundary shear and uplift. For this channel, a boundary shear method as presented in EM 1110-2-1601," Hydraulic Design of Flood Control Channels," U.S. Army Corps of Engineers (COE), 1970 was used. This design was then compared to a velocity method for reasonableness.

The general procedure used was as follows:

1. Determine the mean channel velocity. The mean channel velocity was determined using Mannings equation. A trapizoidal channel with 2(H):1(V) side slopes and 13 foot bottom width was used as given in the plans provided by Olin. For about 16,000 feet upstream of the site, the channel is reported as having a uniform slope of 8.1 feet per mile in "Cayuga Creek Water Quality Study" by Acres American, 1975. This value was used in the velocity determination. Two Manning roughness coefficients were used; one value for a stone-lined channel in good condition and one for a stone-lined channel in poor condition. Various depths of flows were then assumed to determine the corresponding flow quantities and velocities. Based on these determinations, a mean channel velocity of 6.7 feet per second was selected for design purposes. The corresponding flow quantities for a channel in good and poor condition are 2770 and 2080 cubic feet per second. These quantities correspond to at least a 500 year return period as reported in the "Flood Insurance Study for Cayuga Creek," Federal Emergency Management Agency, 1982. Designing for such a lengthy return period is unusually conservative but considering the facility being protected and possible increase in development in the watershed above the site with a corresponding increase in flood volumes, it was deemed appropriate. Also there would not be any significant cost difference for the rip-rap using this return period and a lesser return period because rip-rap thickness would still be the same.

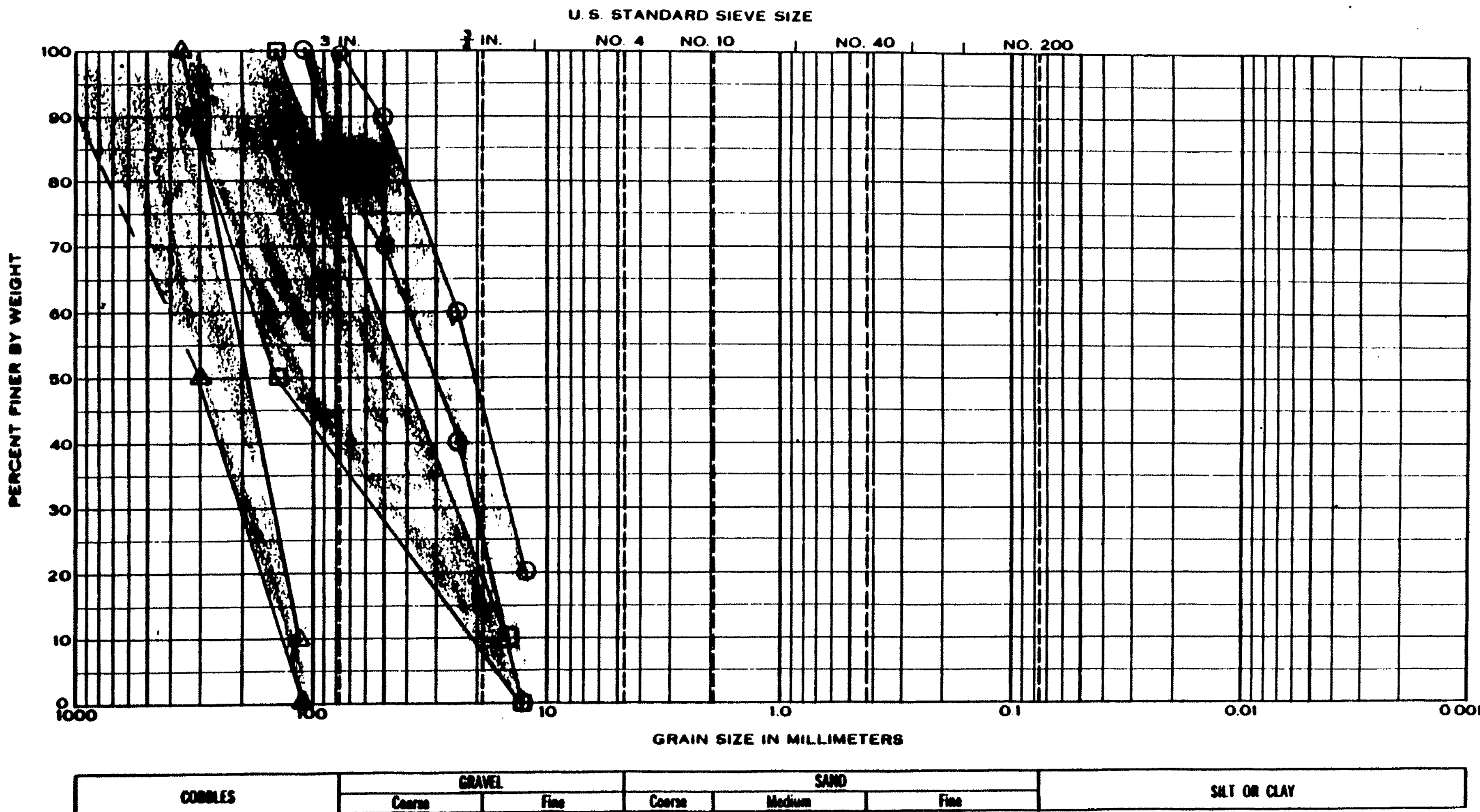
2. Determine average boundary shear,

3. Correct average boundary shear for effect in bend.

4. Determine average stone size, D_{50}.

5. Compare result of COE method with velocity - based methods.

6. Determine rip-rap gradation.



Sample No.	Elev or Depth	Classification	NatWC	LL	PL	PI	
○—○		TYPE I 02200-2					
□—□		Light Riprap Spec					
△—△		Medium Riprap Spec					

GRADATION CURVES	Project
	Date

Woodward-Clyde Consultants

Determination of Minimum Velocity at Which Medium Riprap Should Be Used

Work COE procedure in reverse

Medium riprap has median weight, W_{50}, of at least 100 lbs (NYDOT Spec 620)

Corresponding diameter, D_{50}, is about 1 ft

Riprap design shear, $\bar{\tau}_b = 0.04 (\gamma_s - \gamma) D_{50}$

$= 0.04 (165 - 62.4) 1$

$\tau_b = 4.104$

where γ_s = unit weight of stone, pcf

γ = unit weight of water, pcf

Average boundary shear, $\bar{\tau}_o = \tau_b / 2$

$= 2.052$

$\bar{\tau}_o = \gamma R S$

where R = hydraulic radius

S = slope of channel = $1/650$

$R = \tau_o / \gamma S$

$= 2.052 / (62.4)(1/650)$

$= 21.375$

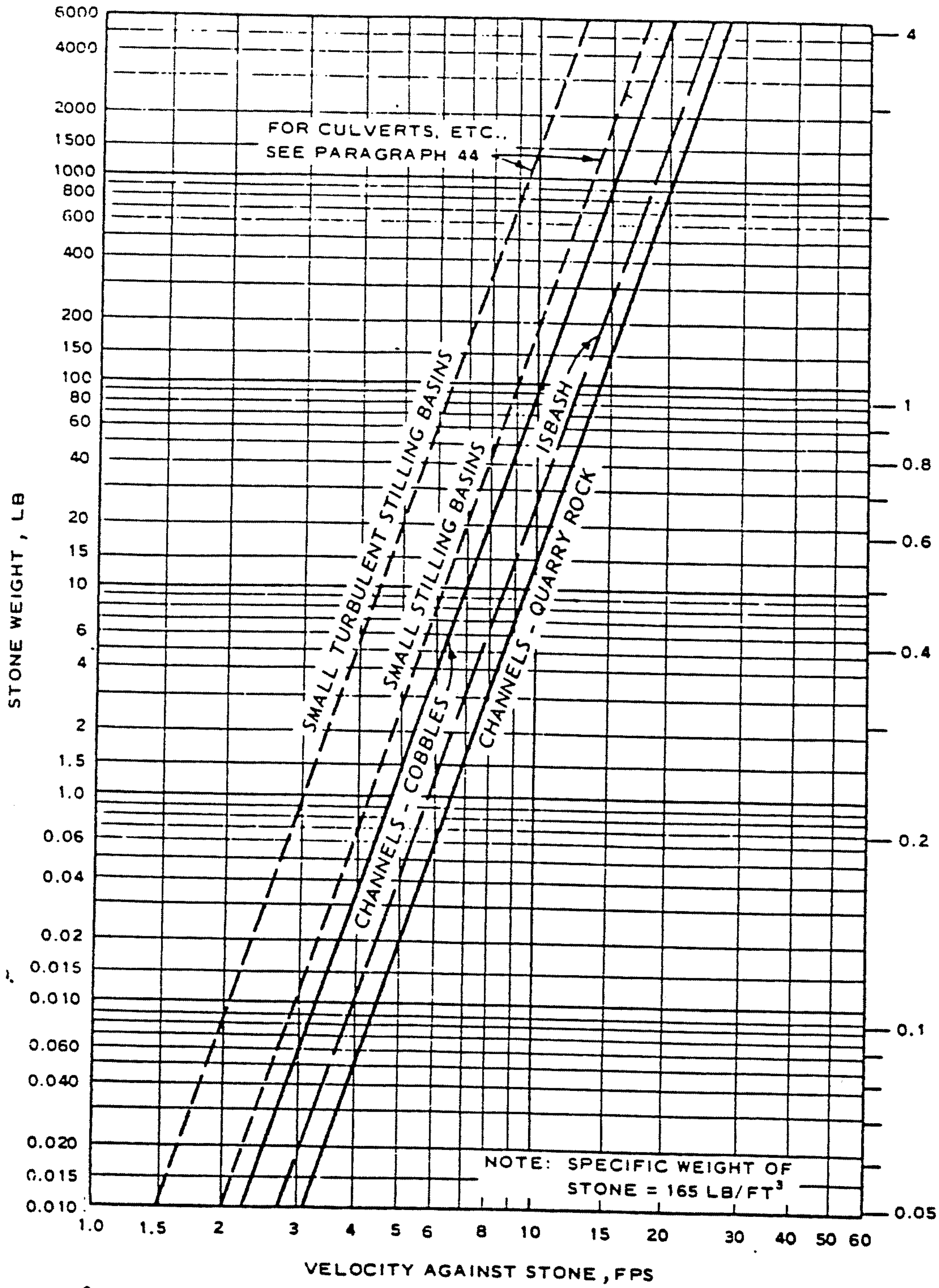
Velocity in Manning equation is proportional to $R^{2/3}$. Using value V of 6.71 and correspond R of 6.43 from riprap design calcs

$$\frac{V_1}{V_2} = \frac{R_1^{2/3}}{R_2^{2/3}} \qquad \frac{6.71}{V_2} = \frac{(6.43)^{2/3}}{(21.375)^{2/3}} \qquad V_2 = 15 \text{ fps}$$

Thus medium riprap should be used if velocity will exceed 15 fps. From the chart on pg 3, this value corresponds to that for channels lined with quarry rock. For round cobbles, say from a sand pit, the velocity is about 10 fps. Since the 500 yr velocity in Cayuga Creek is 6.7 fps and 100 yr velocity about 6 fps, my judgement is that medium riprap is also overkill.

By: BZ Date: 2/7/90 Subject: Riprap Design for Cayuga Creek Sheet 3 of 3

Chkd. by: RDS Date: 2/7/90 Job No.: 89C2857



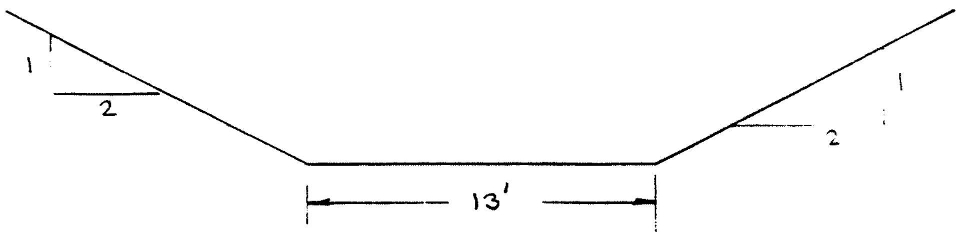
From "Hydraulic Design of Rock Riprap,"
F B Campbell, WES MP 2-777
Feb 1966

STABLE ROCK SIZE

By: JBD Date: 1/4/90 Subject: Riprap Requirements for Cayuga Creek Sheet 1 of 5
Chkd. by: RDS Date: 1/11/90 Job No.: 8962857

Design Parameters for Riprap Design

Channel Dimensions - Given on plans submitted by Olin



Channel Slope - 8.1 ft per mile. Reported average gradient for 16000 ft upstream of site in "Cayuga Creek Water Quality Study" by Acres American, dtd Aug 1975, pg 7 and B-3

Mannings Roughness Coefficient - Values of 0.03 and 0.04 used which correspond to recommended values for canals and ditches with rough stony beds and weeds on earth banks; winding natural stream channels with stones and some weeds. These values consider stream improvements which increase water velocities over those in the present unimproved channel

Flood Volumes - Obtained from "Flood Insurance Study for Cayuga Creek" prepared by Federal Emergency Management Agency for City of Niagara Falls and dtd Sep 1982.

Return Period, yrs	Peak Volume, cfs
10	950
50	1450
100	1650
500	2100

Return period of 500 yrs used

Procedure Used: As presented in EM 1110-2-1601, "Hydraulic Design of Flood Control Channels", US Army Corps of Engineers, dtd July 1970

By: CB Date: 1/4/90 Subject: Riprap Requirement for Cayuga Crk. Sheet 2 of 5
Chkd. by: RDS Date: 1/11/90 Job No.: 89C28C

D, ft	A, ft²	R, ft	Q, cfs n = 0.03	Q, cfs n = 0.04	$\overline{V}$, fps n = 0.03	$\overline{V}$, fps n = 0.04	τ_o, psf	τ_b [illegible] τ_o psf	D_{50}, ft
5.0	115.	3.25	490	367	4.26	3.19	0.312	0.62	
6.5	169.	4.01	829	621	4.91	3.67	0.385	0.77	
7.5	210.	4.52	1116	836	5.31	3.98	0.434	[illegible]	
8.0	232	4.75	1275	956	5.49	4.12	0.456	[illegible]	
10.0	330.	5.72	2051	1538	6.22	4.66	0.549	1.10	
11.0	385.	6.19	2496	1872	6.48	4.86	0.594	1.19	
11.5	414. ✓	6.43 ✓	2776 ✓	2081 ✓	6.71 ✓	5.02	0.617	[illegible]	0.31 ✓

Recommended minimum layer thickness is 1.5 D_{50} or 12 inches whichever is more.

D = depth of flow in channel

A = cross-sectional area of flow

R = hydraulic radius

$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$ (Manning Equation)

n = roughness coefficient

S = slope of bottom channel = (1/650)

$\overline{V}$ = average velocity

$\overline{\tau_o}$ = average boundary shear stress = $\gamma R S$ EM 1110-2-1601, USCE, "Hydraulic Design of Flood Control Channels"

τ_b = maximum boundary shear stress in bend

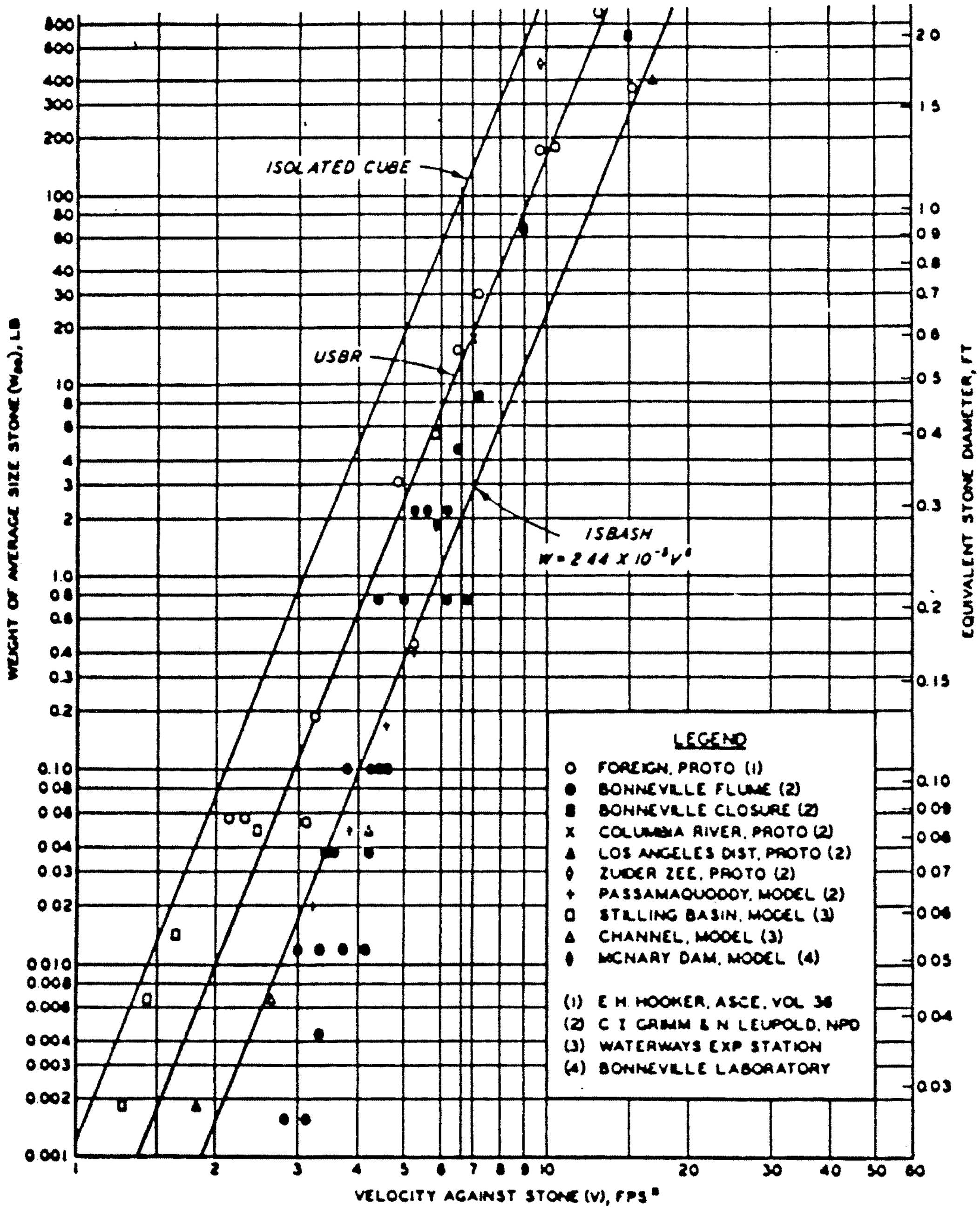
D_{50} = average stone diameter = $\tau_b / 0.04 (\gamma_s - \gamma)$

γ_s = unit weight of stone = 165 pcf

By: CGL Date: 1/4/90 Subject: Riprap Placement for ... Sheet 3 of 5

Chkd by: RDS Date: 1/10/90 Job No.:

Other design criteria



NOTE: SPECIFIC WEIGHT OF ROCK = 165 LB/CU FT.
*FOR STONE PROTECTION BELOW ENERGY DISSIPATORS USE AS AVERAGE VELOCITY ACROSS END SILL.

From EM 1110-2-1601, USCE
"Hydraulic Design of Flood Control Channels"

VELOCITY VS STONE WEIGHT

SEE TEXT PAGE 36

where K, which is termed the *discharge factor*, depends for its value on the ratio of the depth of water to some other linear dimension of the cross section.

TABLE 19. HORTON'S VALUES OF n TO BE USED WITH THE MANNING FORMULA

Surface	Best	Good	Fair	Bad
Vitrified sewer pipe	{0.010, 0.011}	0.013*	0.015	0.017
Common-clay drainage tile	0.011	0.012*	0.014*	0.017
Glazed brickwork	0.011	0.012	0.013*	0.015
Brick in cement mortar; brick sewers	0.012	0.013	0.015*	0.017
Neat cement surfaces	0.010	0.011	0.012	0.013
Cement mortar surfaces	0.011	0.012	0.013*	0.015
Concrete pipe	0.012	0.013	0.015*	0.016
Wood-stave pipe	0.010	0.011	0.012	0.013
Plank flumes:				
Planed	0.010	0.012*	0.013	0.014
Unplaned	0.011	0.013*	0.014	0.015
With battens	0.012	0.015*	0.016	
Concrete-lined channels	0.012	0.014*	0.016*	0.018
Cement-rubble surface	0.017	0.020	0.025	0.030
Dry-rubble surface	0.025	0.030	0.033	0.035
Dressed-ashlar surface	0.013	0.014	0.015	0.017
Semicircular metal flumes, smooth	0.011	0.012	0.013	0.015
corrugated	0.0225	0.025	0.0275	0.030
Canals and ditches:				
Earth, straight and uniform	0.017	0.020	0.0225*	0.025
Rock cuts, smooth and uniform	0.025	0.030	0.033*	0.035
jagged and irregular	0.035	0.040	0.045	
Winding sluggish canals	0.0225	0.025*	0.0275	0.030
Dredged earth channels	0.025	0.0275*	0.030	0.033
Canals with rough stony beds, weeds on earth banks	0.025	0.030	0.035*	0.040
Earth bottom, rubble sides	0.028	0.030*	0.033*	0.035
Natural stream channels:				
1. Clean, straight bank, full stage, no rifts or deep pools	0.025	0.0275	0.030	0.033
2. Same as 1, but some weeds and stones	0.030	0.033	0.035	0.040
3. Winding, some pools and shoals, clean	0.033	0.035	0.040	0.045
4. Same as 3, lower stages, more ineffective slope and sections	0.040	0.045	0.050	0.055
5. Same as 3, some weeds and stones	0.035	0.040	0.045	0.050
6. Same as 4, stony sections	0.045	0.050	0.055	0.060
7. Sluggish river reaches, rather weedy or with very deep pools	0.050	0.060	0.070	0.080
8. Very weedy reaches	0.075	0.100	0.125	0.150

* Values commonly used in designing.

For trapezoidal channels (including rectangular and triangular sections)

$$K = \frac{1.486\left(\frac{1}{x} + z\right)^{2/3}}{\left(\frac{1}{x} + 2\sqrt{1 + z^2}\right)^{2/3}} \qquad (155b)$$

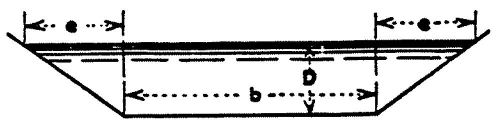


FIG. 73. Trapezoidal canal section.

where $x = D/b$ = ratio of depth of water to bottom width of channel (Fig. 73) and $z = e/d$ = side slopes of channel, ratio of horizontal to vertical. Table 20 contains values of K corresponding to D/b for various side slopes.

From "Civil Engineering Handbook", Mc Graw. Hill, 1950.

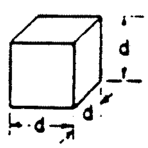
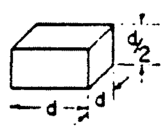
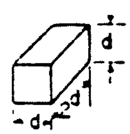
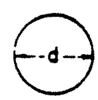
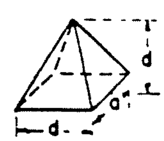
FIGURE 620-1 STONE FILLING GRADATION REQUIREMENTS

Stone Filling Item	See Notes	Stone Size[1]	Percent of Total by Weight
Fine	2, 3, 4	Smaller than 8 ins. Larger than 3 ins. Smaller than No. 10 Sieve	90 - 100 50 - 100 0 - 10
Light	2, 3, 4	Lighter than 100 lbs. Larger than 6 ins. Smaller than ½ in.	90 - 100 50 - 100 0 - 10
Medium	2, 4	Heavier than 100 lbs. Smaller than 4 ins.	50 - 100 0 - 10
Heavy	2, 4, 5	Heavier than 600 lbs. Smaller than 6 ins.	50 - 100 0 - 10

NOTES:

1. Stone sizes, other than weights, refer to the average of the maximum and minimum dimensions of a stone particle as estimated by the Engnieer.
2. Materials shall contain less than 20 percent of stones with a ratio of maximum to minimum dimension greater than three.
3. Air-cooled blast furnace slag, cobbles or gravel having at least one fractured face per particle are acceptable substitutes for stone under these items, provided that soundness and gradation requirements are met.
4. Materials shall contain a sufficient amount of stones smaller than the average stone size to fill the spaces between the larger stones.
5. Heavier gradings of this item may be required on some projects, in which case the requirements will be stated on the plans or in the proposal.

TABLE 620-2

Specified Weights and Sizes	Approximate Shape				
				 Sphere	
600 lbs.	d=18 ins.	d=23 ins.	d=15 ins.	d=23 ins.	d=27 ins.
300 lbs.	d=15 ins.	d=18 ins.	d=12 ins.	d=18 ins.	d=21 ins.
150 lbs.	d=12 ins.	d=15 ins.	d= 9 ins.	d=15 ins.	d=17 ins.
100 lbs.	d=10 ins.	d=13 ins.	d= 8 ins.	d=13 ins.	d=15 ins.
d=8 ins.	50 lbs.	25 lbs.	100 lbs.	25 lbs.	16 lbs.
d=6 ins.	20 lbs.	10 lbs.	40 lbs.	10 lbs.	7 lbs.

By: JBD Date: 1/9 Subject: Riprap Design for Cayuga Creek. Sheet ____ of ____
Chkd. by: ______ Date: ______ Job No.: 89C2857

4" Median Gradation for Riprap

Percent Lighter by Weight	Limits of Stone Weight, lb	Equivalent Diameter, in
100	6.5 - 16	5 - 7
50	3.0 - 5.0	4 - 4.5
15	0.1 - 1.0	1.3 - 2.7

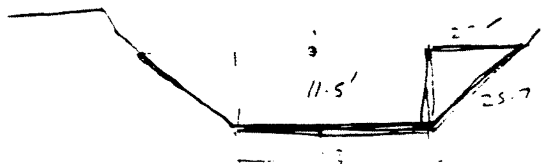
6" Median Gradation for Riprap

Percent Lighter by Weight	Limits of Stone Weight, lbs	Equivalent Diameter, in
100	22 - 54	7.5 - 10.3
50	11 - 16	6 - 7
15	0.4 - 4	2 - 4.5

Minimum Layer thickness for 4" Median Riprap - 12"
6" - 18"

Recommend 4" median gradation which corresponds to NYDOT "Light" riprap.

Page [illegible]



$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2}$$

$$= \frac{1.49}{.03} \cdot 414 \, (6.43)^{2/3} \left(\frac{1}{650}\right)^{1/2}$$

$$= 20562 \cdot 3.45722 \times \frac{1}{25.5}$$

$$= \underline{2788} \ ft^3/sec$$

$A = (13 + 23)\ 11.5$

$= 36 \times 11.5$

$P = 64.4$

$R = \frac{A}{P} = \frac{414.0}{64.43} = 6.43$

529

$Q = 2788 \qquad v = 6.73 \ ft/sec$

$\tau_0 = 62.4 \times 6.43 \times \frac{1}{650} = 0.617 \ psf$

$\tau_b = 2\tau_0 = 1.234 \ psf$

$\gamma_s = 165 \ lbs/ft^3$

$$D_{50} = \frac{1.234}{0.04\,(\gamma_s - \gamma)}$$

$$= \frac{1.234}{4} = \underline{\underline{0.31}} \ ft \sim 4''$$

$1.5\ D_{50} = \underline{\underline{6''}} < 12''$ recommended

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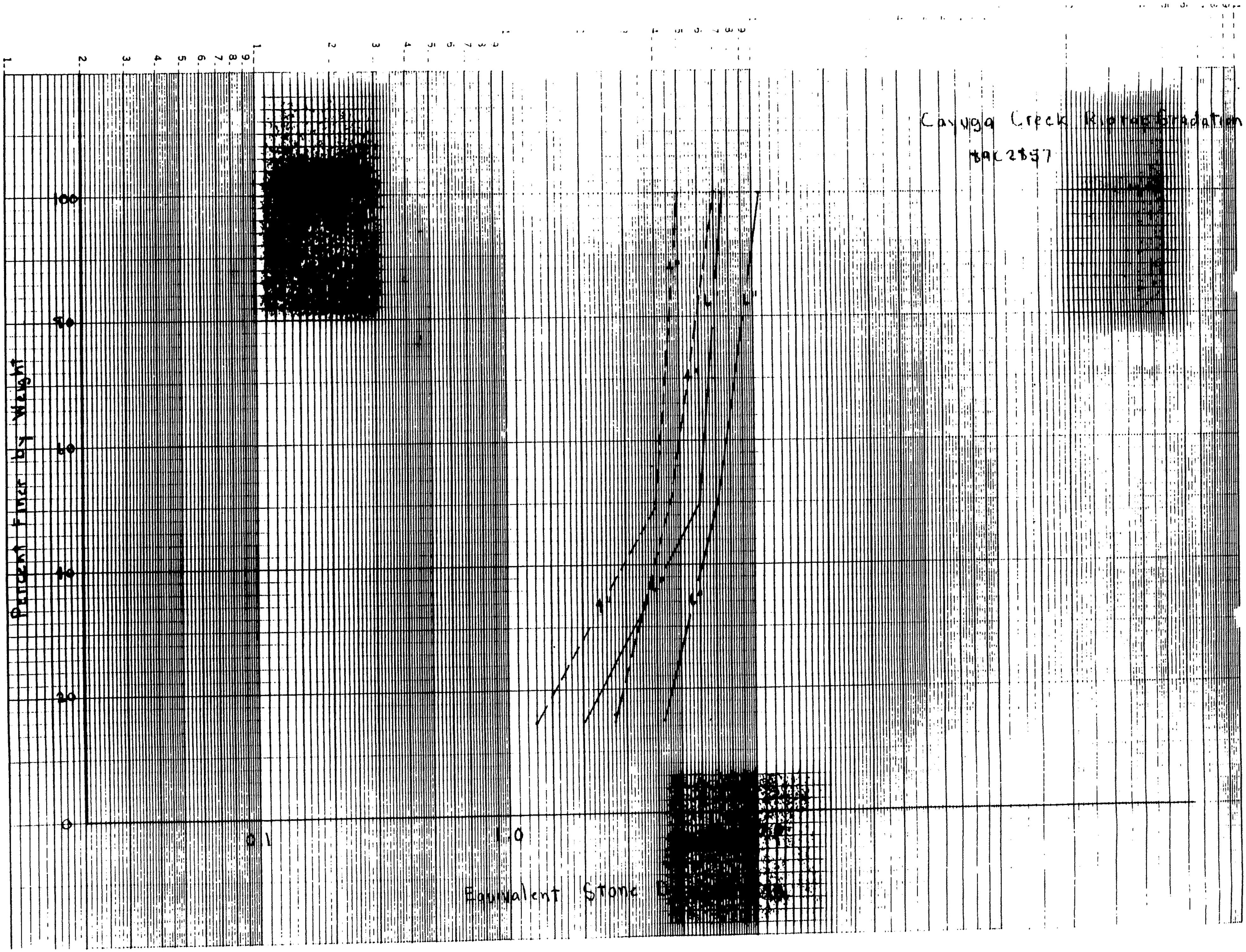

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